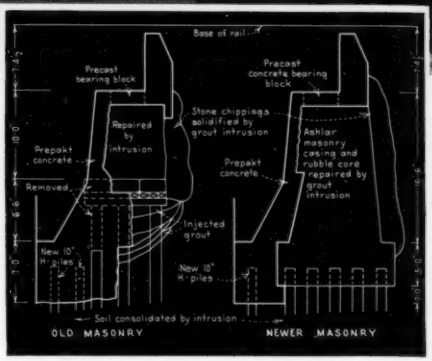
ACI News Letter

Vol. 23 No. 1 JOURNAL of the AMERICAN CONCRETE INSTITUTE September 1951



Above: Railroad masonry bridge abutment and wing wall after rehabilitation with Prepakt Concrete and Intrusion Grout.

Right: Schematic drawing showing areas of old and new masonry repaired and solidified.

OLD ABUTMENTS MADE *Stronger than New!*

The Problem—The stone masonry of the bridge abutments had progressively deteriorated due to seepage, scour, and undermining, to a point where cracking, settling and tipping made rehabilitation imperative.

The Answer—Prepakt Concrete and Intrusion Grout methods and materials restored these abutments to a strength equal to or greater than new—and with higher resistance to future deterioration.

The methods included the solidification of earth and fill under and behind the abutments, solidification of stone within the structure, and refacing.

Special Intrusion-Prepakt materials insured complete filling of all voids and cracks, non-settling in hardening, and near-perfect bond with old masonry or concrete—thus providing a stronger-than-new structure.

Full details on this abutment restoration may be obtained by writing to Intrusion-Prepakt, Inc., Union Commerce Building, Cleveland, Ohio.

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September 1951

On the Cover—One of eleven 13-story reinforced concrete apartment buildings under construction in San Francisco for the Metropolitan Life Insurance Co. Further details on p. 29 of the Journal.

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St. Louis Regional Meeting Announced

A. Carl Weber, director of research of Laclede Steel Co., and chairman of a St. Louis ACI committee, has announced the program for a fall regional meeting to be held at the Sheraton Hotel, St. Louis, Mo., Oct. 30-31, 1951.

A report covering a four year study of experimental continuously reinforced concrete pavement in Illinois will present results obtained with this type of construction designed to reduce or eliminate pumping and faulting at joints with resultant structural damage to concrete pavements under recurring heavy loads.

Of particular interest to users and producers of ready-mix concrete will be a discussion of Midwestern practices in that industry. Economics of high-early strength cement concrete and tests of concrete slabs reinforced with welded wire fabric will also be reported in this session. A new handbook, *Useful Data*, will be reviewed.

The final session will feature construction of multistory concrete tanks, conservation by design and a movie on the use of rail steel. An inspection trip to concrete construction jobs in the St. Louis area will top off the meeting.

The St. Louis committee has asked that hotel reservations be made direct to the Sheraton Hotel, attention W. W. Netz, with the designation that they are for the ACI conference.

William C. E. Becker, consulting engineer, is meeting chairman, Clarence H. Ax, Jones-Kissner Construction Co., is committee secretary and Edward Asche, Construction Materials Co., is treasurer. Carl Chappell, PCA, is in charge of publicity and other subcommittee chairmen are Sidney Bierman, Precon Concrete Products, reservations; Walter Bryan, Engineers' Club of St. Louis, registration; Col. Joseph Vollmer, Fruin-Colnon Contracting Co., finance; W. W. Horner, Horner and Shifrin, program; J. W. Hubler, Washington University, educational contacts; George Rogan, entertainment; and Howard Coleman, Missouri Portland Cement Co., inspection trip. Other committee members with one or more subcommittee assignments are A. H. Baum, building commissioner for St. Louis; John Brouk, Precast Slab and Tile Co.; Neal Campbell, consulting engineer; A. H. Lamack, Laclede Steel Co.; Ralph Teich, H. A. Dailey, Inc.; Henry Till; Associated General Contractors of St. Louis; and Robert Willis of PCA. ACI President Harry F. Thomson, Materials Service Corp., is ex officio member of the Committee.

Program, Regional Meeting
Sheraton Hotel, St. Louis, Mo.

TUESDAY, OCTOBER 30

9:30 a.m.—REGISTRATION—No Charge

($10.00 entertainment fee includes admission to luncheon, cocktail party, dinner and inspection trip)

11:00 a.m.—FIRST TECHNICAL SESSION

Chairman—
ACI Past President FRANK H. JACKSON
Principal Engineer of Tests
Bureau of Public Roads

Four Year Report on the Experimental Continuously Reinforced Concrete Pavement in Illinois

J. D. LINDSAY, Engineer of Materials,
Illinois State Highway Dept.

12:30-2:00—INFORMAL LUNCHEON

Chairman—
A. CARL WEBER
Director of Research, Laclede Steel Co.

Introduction of officers, directors and guests of ACI

Luncheon Speaker A. H. BAUM, Building Commissioner St. Louis, Mo.
Chairman, Building Officials Conference of America
(Ticket included with entertainment fee)

1:00-3:00 p.m.—LADIES LUNCHEON

Style Show and tour of St. Louis
Arrangements to be made at time of registration

2:00-5:00 p.m.—SECOND TECHNICAL SESSION

Chairman—
ACI President HARRY F. THOMSON
Manager, Ready-Mixed Concrete Div.,
Material Service Corp.

Ready-mix Concrete Practices in the Middlewest

A. W. BRUST, Dept. of Civil Engineering,
Washington University

Economics of High Early Strength Cement Concrete

M. W. JOHNSON, Pittsburgh Testing Laboratories,
and L. A. WAGNER, Director of Engineering and Research, Missouri Portland Cement Co.

Instrumentation and Strain Measurement in Welded Wire Fabric Reinforced Concrete Slabs

E. W. CARLTON and
JOSEPH SENNE, Dept. of Civil Engineering,
Missouri School of Mines and Metallurgy

Useful Data—A New Handbook for the Concrete Designer
RAYMOND C. REESE, Consulting Engineer
Toledo, Ohio

6:30 p.m.—COCKTAIL HOUR

7:30 p.m.—DINNER—INFORMAL DRESS

Chairman—
WILLIAM C. E. BECKER,
Consulting Engineer, St. Louis, Missouri

Backward or Forward
COLONEL F. W. GREEN, Retired President,
St. Louis-Southwestern Railway Co.

WEDNESDAY, OCTOBER 30

10:00 to 12:00—THIRD TECHNICAL SESSION

Chairman—
ACI Past President H. J. GILKEY, Head,
Dept. of Applied Mechanics, Iowa State College

Field Practices in the Construction of Multiple Story Concrete Tanks
EVANGELOS J. CRITZAS, Construction Engineer,
Bosari Tank Corp.

Analysis of Prestressed Concrete Structures
ROBERT B. B. MOORMAN, Dept. of Civil Engineering, Univ. of Missouri

Conservation by Design
O. W. IRWIN, President, Rail Steel Bar Assn.

Rail Steel in the World Today
Motion Picture with LOWELL THOMAS, narrator

1:30-4:30 p.m.—INSPECTION TRIP

An inspection of outstanding concrete construction in the St. Louis area. Buses will leave Sheraton Hotel and return.

Reservations should be made direct with the hotel, addressed Mr. W. W. Netz, Sheraton Hotel, St. Louis, Mo., with the advice that the reservation is being made in connection with the ACI regional meeting.

To Clarify Bylaws

ACI's Bylaws are being studied by a special Board committee to develop constructive suggestions for additions, deletions or other changes. The committee, consisting of H. J. Gilkey, Chairman, A. T. Goldbeck and C. E. Wuerpel, has requested this News Letter notice inviting members to submit their suggestions for additions, deletions or other changes to the Secretary's office or to the chairman not later than October 15. Little time will remain after this date for the committee to review and correlate suggestions for its meeting October 29. Therefore, members are requested to submit their suggestions as promptly as possible and to confine their letters to the subject of Bylaws changes only. Letters received at the Secretary's office will be forwarded promptly to the chairman, H. J. Gilkey, Head, Department of Theoretical and Applied Mechanics, Iowa State College, Ames, Iowa.

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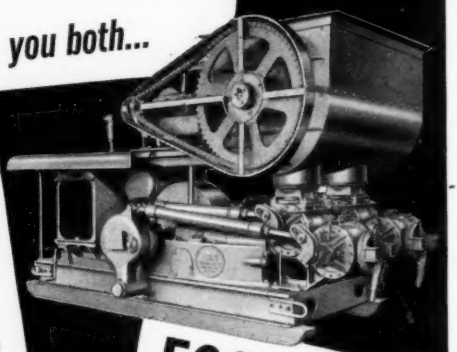

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- Pumpcrete will handle stiffer mixes
- Pumpcrete reduces loss of slump between the pump and the point of placement
- Pumpcrete will fill overbreaks higher . . . tighter than any other method . . . up to 20 feet
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ECONOMY

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- With horizontal layer method specified, Pumpcrete is the only mechanical method that can be used . . . and is best for advancing slope, too.
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ST. LOUIS: GATEWAY TO THE WEST

St. Louis of today, Missouri's largest city and the nation's eighth largest, still takes pride in its varied history. The real establishment of St. Louis came in 1764 when Maxent, Laclede and Co. of New Orleans began to exploit exclusive rights to the Indian trade in the Missouri Valley and all the country west of the Mississippi.

Within five years, St. Louis became the fur trading center of the west. Two years later there was an influx of new settlers, merchants and adventurers. Following the Revolution, St. Louis and river traffic boomed. The growing fur trade, which by 1804 was valued at $203,750 annually, made St. Louis the center of wealth and culture in the upper river valley.

When Napoleon sold the Louisiana Territory to the United States in 1803 the westward movement began with St. Louis as the gateway and capital of the vast, rich, new half of a continent. Between 1804 and 1806, the historic expedition of Lewis and Clark set forth from its base at St. Louis; in 1809, the famous Missouri Fur Co. was organized and, in 1822, John Jacob Astor opened a St. Louis branch to compete for the $300,000 per year fur trade.

Political events in Europe provided St. Louis with its third wave of frontiersmen as large numbers of German professional men and scholars, skilled tradesmen and artisans, dissatisfied with conditions in their homeland, began moving to Missouri between 1832 and 1850. These stable and cultivated people brought to the midwestern community new interest in education, art and industry which contributed much to its development.

In 1904, the attention of the entire nation was focused on modern St. Louis and its great Louisiana Purchase Exposition, the precursor of the glittering "World's fairs" of our own day. Celebrating the growth of St. Louis and the Middle West, the exposition presented a panorama of not only the nation but of the world in that era of prosperity and plenty. The nations of Europe and Asia sent lavish displays to the Fair to make it a vast, opulent spectacle.

Today a hub city for transportation, manufacturing, commerce and finance as well as the dominant center in a wealthy agricultural area, St. Louis is the true product of its past, combining a pride in its historic heritage with the spirit of progress which made it great.

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2. Civil engineering or architectural engineering or equivalent.
3. Confidence in your ability to show others the value of your sincere convictions —or confidence in your ability to learn to do so.

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CONSTRUCTION MACHINERY DIVISION
CHAIN BELT COMPANY, *MILWAUKEE, WISCONSIN*

"The 25's"

Special note has been taken of American Concrete Institute records over a quarter of century ended in 1950 showing outstanding contributions in that period toward membership growth.

More than 1300 Members, current or former, are sponsors of recruits to ACI's effort in the progress of knowledge of concrete and its application to engineering practice.

Twenty-five of the most successful proselyters are listed for special distinction for adding 25 or more Members each to our roll during 25 years.

The Board of Direction expresses thanks for and appreciation of the accomplishment of those whose names are appended. Through others they have extended, widened and deepened their own adherence to the cause of better construction.

Allan, W. D. M.
Blanks, Robert F.
Clair, Miles N.
Davis, Raymond E.
Dovali Jaime, Alberto
Emerson, H. B.
Gilkey, Herbert J.
Gonnerman, Harrison F.
Kennedy, Henry L.
McCarthy, James A.
McMillan, Franklin R.
Menefee, Ferdinand N.
Morales, Rene P.
Morgan, Newlin D.
Morgan, Newlin D., Jr.
Price, Walter H.
Richart, Frank E.*
Savage, John Lucian
Stanton, Thomas E.
Tuthill, Lewis H.
Upson, Maxwell M.
Voss, Walter C.
Walker, Stanton
Way, W. F.
Wuerpel, Charles E.

*Deceased

Honor Roll

February 1—July 31, 1951

At the halfway mark in this year's Honor Roll, Newlin D. Morgan, Sr., leads with 29 credits. Newlin D. Morgan, Jr., holds second place with 11½ credits and Eddy Hernandez is third with 8½ credits. There is still time for **you** to take your place among the top Member-getters.

Newlin D. Morgan, Sr. (Ill.)	29
Newlin D. Morgan, Jr. (Mich.)	11½
Eddy Hernandez (La.)	8½
J. E. Jellick (Calif.)	6½
Herbert J. Gilkey (Ia.)	6
L. G. Farrant (Fla.)	5½
Samuel Hobbs (Calif.)	5½
Oscar A. Nunez G. (Venezuela)	5½
Alexander Klein (Calif.)	4
James A. McCarthy (Ind.)	4
Fernan Rodriguez-Gil (Venezuela)	3½
George D. Youngclaus (Calif.)	3½
Frank Kerekes (Ia.)	3
Alvin C. Loewer, Jr. (Pa.)	3
Rafael Ruiz (Guatemala)	2½
Oskar Schreier (N. Y.)	2½
Donald D. Agrimson (S. D.)	2
J. F. Baxter (N. Y.)	2
Robert G. Deitrich (Md.)	2
Frank L. Ehasz (N. Y.)	2
Oliver J. Julian (Mass.)	2
T. C. Kavanagh (Pa.)	2
Thomas J. Reading (S. D.)	2
C. P. Siess (Ill.)	2
Howard Simpson (Mass.)	2
Ivan M. Viest (Ill.)	2
Jose A. Vila (Cuba)	2

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Dorena, Oregon
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U. S Corps of Engineers
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...that

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Hot Springs, South Dakota
Proof...of Protex leadership in Air Entraining Agents!

Ft. Gibson Dam
U.S Corps of Engineers
Muskogee, Oklahoma
Proof...that Protex gives consistent control!

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Name.....

Address.....

City.....*State*.....

Who's Who This Month

R. R. Martel

"Earthquake Resistant Design Considerations," p. 1, are summarized by R. R. Martel, professor of structural engineering at California Institute of Technology. Prof. Martel was graduated from Brown University in 1912, and in addition to teaching is a consulting engineer on bridges, buildings and other structures. His special interest is earthquake resistant structures.

Whitney, Anderson and Salvadori

"Comprehensive Numerical Method for the Analysis of Earthquake Resistant Structures," p. 5, is a joint contribution of Charles S. Whitney, Boyd G. Anderson and Mario G. Salvadori.

Mr. Whitney, an ACI Director and well known for his contributions to reinforced concrete design, particularly in thin shell construction and other arched structures, is a partner in the consulting engineering firm of Ammann and Whitney, New York and Milwaukee.

Honors to Mr. Whitney are many, including the ACI Wason Medal for most meritorious paper for his 1932 "Plain and Reinforced Concrete Arches," the Alfred E. Lindau award in 1951 "in recognition of his many contributions to reinforced concrete design practice," the Fuertes Graduate Medal of Cornell University twice, and the J. James R. Croes Medal by ASCE.

He previously was a member of ACI's Board of Direction, 1945-46, and is at present chairman of Committee 312, Plain and Reinforced Concrete Arches, and Chairman of Committee 326, Shear and Diagonal Tension (a joint committee with ASCE), a member of Committee 214, Evaluation of Results of Compressive Tests of Field Concrete.

Since 1922 Mr. Whitney has been a consulting and designing engineer specializing in city planning, design and supervision of buildings, bridges, sewer and water systems and other municipal improvements.

Boyd G. Anderson is chief designing engineer with Amman and Whitney. Previously he was an engineer with Charles S. Whitney, consulting engineer, and field engineer on buildings, canal locks and piers in the designing engineer section of the Bureau of Yards and Docks, U. S. Navy, on timber, steel, concrete and precast concrete aircraft hangars. He graduated in civil engineering from the University of Wisconsin in 1936.

Mario G. Salvadori is a consultant to Ammann and Whitney and professor of civil engineering at Columbia University. He was an instructor and assistant professor at the University of Rome 1930-38 and consultant to the Institute for Application of Calculus, Rome, and secretary of the civil engineering section of the Italian Research Council. He later was a time and motion study engineer for the Lionel Corp. and special consultant to the Manhattan District Project.

He received doctorates in civil engineering and mathematics from the University of Rome and was a research fellow in photoelasticity at University College, London. He is the author of papers on theory of structures, reinforced concrete, elasticity and photoelasticity in several Italian, English and American technical periodicals. He is a member of ASCE and of the Society for Experimental Stress Analysis.

John J. Gould

John J. Gould, whose "Multistory Buildings Designed to Resist Earthquakes," p. 29, reflects recent work in San Francisco, has been a consulting engineer since 1940. Work of his firm includes such buildings as the 300 ft high Oakland Long Distance Exchange Building, Army installations, numerous buildings for the F. W. Woolworth Co., factories and warehouses for the National Lead Co. as well as the buildings for the Parkmerced Housing Development.

A native of Switzerland, Mr. Gould is a graduate of the Engineering School there. After coming to the United States he worked for a time in New York before going to San Francisco where he became design engineer for L. H. Nishkian. Following the Long

Continued on p. 12

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CEMENT and CONCRETE COMPRESSION TESTING MACHINE

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Meets all A.S.T.M. requirements, provides for testing concrete cylinders up to 8″ x 16″, and also is equipped for testing concrete building blocks up to 12″ wide x 18″ long.

Single ram construction, increased ram stroke, and fast operation facilitate alternate testing of cylinders and blocks.

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Eddystone Division, Baldwin-Lima-Hamilton Corp., Philadelphia 42, Penna.
In Canada: Peacock Bros., Ltd., Montreal, Quebec

Beach earthquake in 1933 he became the senior structural engineer for the State of California where he was primarily concerned with the earthquake resistant construction of new and existing school buildings. In 1935 he became chief structural engineer for the San Francisco Bay Exposition Co. in full charge of the structural features of the permanent construction on Treasure Island as well as the temporary construction used for the Exposition.

John A. Murlin

"Lightweight Concrete for Lower Construction Costs," p. 37, introduces John A. Murlin, chief structural engineer for George L. Dahl, Architects and Engineers, Dallas, Texas. A mechanical engineering graduate of the Massachusetts Institute of Technology in 1926, he now is a registered professional engineer.

His experience includes aircraft structures, hydraulic machinery, production control, acoustics and heat transfer with nationally known firms in Ohio, Pennsylvania and New York. He has been a member of ACI since 1949.

Alfred L. Parme and George H. Paris

The timely paper "Designing for Continuity in Prestressed Concrete Structures," p. 45, is the work of Alfred L. Parme and George H. Paris of the Portland Cement Assn.

Mr. Parme will be remembered for his paper "Solution of Difficult Structural Problems by Finite Differences," in the November 1950 JOURNAL. He is an engineer with the Structural and Railways Bureau, Portland Cement Assn. A member of ACI Committee 314, Rigid Frame Bridges, he is also a member of the ASCE Committee on thin shell design and the author of several papers on structural design.

After graduation from Cornell University in 1935 Mr. Parme was employed by the Phoenix Engineering Co. on the design of hydraulic structures. Joining the Corps of Engineers in 1937 he engaged in the structural design of flood control appurtenances such as conduits, intakes and dams. In 1940 he joined the PCA staff, collaborating in the preparation of structural data and reports on the design of reinforced concrete structures.

Mr. Paris has been a structural engineer with the Structural and Railways Bureau of the Portland Cement Assn. for the past five years. He was for an equal period with the bridge department of the Burlington Railroad. He graduated from the University of Illinois with a BS in civil engineering.

Leslie I. Neher

Leslie I. Neher, consulting engineer, Marion, Ind., is the author of "Pumice—Lightweight Aggregate," p. 65.

Following World War II, Mr. Neher became interested in pumice lightweight aggregates for concrete, insulation fill and abrasive applications. As sales engineer for a New Mexico producer, he introduced pumice in Indiana, Michigan and Illinois. He was general manager of a pumice producer in New Mexico and was active in mining, grading, research and sales; made many field trips in the study of improved methods of monolithic and masonry construction applicable to lightweight aggregate concrete and designed concrete mixes and structural details using pumice aggregate and concrete for grain drying and curing structures.

During World War II, Mr. Neher was a Colonel in the Air Force and was associated with the aircraft technical training program of very heavy bombardment. He was general manager of the Korea Electric Power Company, Seoul, Korea, with the main effort in the rehabilitation of coal mines, railways, buildings and equipment of hydroelectric and powdered-coal generating plants.

Mr. Neher is a member of ACI, ASTM, Society of Metals, New Mexico Society of Professional Engineers and the U. S. Air Force Reserve.

George L. Kalousek and Milton Adams

"Hydration Products Formed in Cement Pastes at 25 to 175 C," p. 77, reports the results of experimental work at the University of Toledo, Ohio, by the authors.

George L. Kalousek is a member of the research staff of Owens-Illinois Glass Co. working on insulating materials and is director of research fellowships in silicate chem-

Continued on p. 27

Specify Bars by Number
rather than by fractional size

The Bethlehem Concrete Reinforcing Bar is now identified by numbers from #2 to #11, in accordance with Simplified Practice Recommendation 26-50 of the U. S. Dept. of Commerce. Each number indicates approximately the eighths-of-an-inch in the nominal diameter of the bar. Bars #9, #10 and #11 are equal in weight and nominal area to the old type 1 in., 1⅛ in., and 1¼ in. square bars. Bar #2 is a plain round bar. All other sizes are deformed bars with the bar number plainly branded at regular intervals to facilitate identification.

All sizes of the Bethlehem Bar are rolled as round sections. As no change has been made in either the standard unit weights or nominal cross-sectional areas, there is no need for you to make any changes in design, as far as they concern bar sections.

The Bethlehem Reinforcing Bar is high in quality because it is made from new-billet steel which meets ASTM Specification A15. The bar provides a firm bond in concrete because its high, uniformly spaced lugs meet the deformation requirements of ASTM Specification A305. These lugs also minimize slippage at working loads, and they tend to eliminate the formation of wide tensile cracks in the concrete.

To be sure of a dependable reinforcing bar, ask for the Bethlehem Bar. And take the easy way—specify your bar sizes by number.

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STRUCTURAL ENGINEER:
Alfred Benesch & Associates
CONTRACTOR:
Lloyds Builders, Inc.

New apartment building at 3410 Lake Shore Drive, Chicago. 11,000 cubic yards of Darex AEA concrete. Scheduled for completion October, 1951.

On job after job these facts have been proved: Darex AEA concrete places easier and faster, finishes better, has better surface texture, is more durable when exposed to freezing and thawing.

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Cement Products Division

DEWEY & ALMY Chemical Company

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25-Year Members

As the JOURNAL begins another Volume year, a salute is appropriate to those long-time members who have held their memberships through good times and bad and who, year after year, have remained a nucleus of ACI's continuation and growth. These individuals and firms have been, according to ACI records, members for 25 years or more. Three charter members are listed: Henry C. Turner, Alpha Portland Cement Co. and Lehigh Portland Cement Co.

Name	Date
AIREY, EDWIN	1925
ALIN, A. L.	1920*
ALLENTOWN PORTLAND CEMENT CO.	1910*
ALPHA PORTLAND CEMENT CO.	1905†
AMERICAN CAN CO.	1914*
AMERICAN CONCRETE PIPE ASSN.	1922
ANTI-HYDRO WATERPROOFING CO.	1921*
ARIZONA HIGHWAY DEPT.	1926
BABCOCK, HAROLD A.	1924
BAKKER, JAN A.	1920*
BALLOU, HENRY W.	1921*
BAUER, EDWARD E.	1923
BECKER, WM. C. E.	1922
BERG, U. T.	1926
BERKSHIRE GRAVEL, INC.	1925
BERTIN, RENE L.	1921*
BETHLEHEM STEEL CO.	1924
BOGUE, ROBERT H.	1925
BORCHARD, E. K.	1924
BRASSERT, WALTER O.	1919*
BRAY, WILLIAM T.	1926
BRUNDAGE, AVERY	1925
BRUNNIER, HENRY J.	1920*
BUFFALO SLAG CO.	1922
BUSSE, FRANK A.	1925
BUTLER, MORGAN R.	1922
CAMPBELL, STACY A.	1924
CANADA CEMENT CO., LTD.	1916*
CARNWATH, JAMES	1925
CEMENTO DE MIXCOAC, S. A.	1922
CHRISTY, CARL O.	1926
COHEN, A. BURTON	1910*
CONSOLIDATED SAND & GRAVEL, LTD.	1925
CONWELL & CO., E. L.	1908*
CRABBS, AUSTIN	1921*
CRANE, THEODORE	1924
CROSS, HARDY	1924
CUMMINGS, A. E.	1924
CUTLER, MYRTON H.	1925
DARLING, ERNEST H.	1920*
DAVIS, HERBERT A.	1921*
DAVIS, RAYMOND E.	1926
DAVIS, WATSON	1918*
DAY, WILLIAM P.	1925
DETROIT EDISON CO.	1921*
DI STASIO, JOSEPH	1920*
DIXON, DE FOREST H.	1911*
DOCKSTADER, E. A.	1922
DOUTHETT, CLAUDE L.	1926
DOW CHEMICAL CO.	1924
DRESSER, T. P., JR.	1922
DREYER, WALTER	1922
DUQUESNE SLAG PRODUCTS CO.	1917*
DWYER, JOHN R.	1921*
EARLE, G. & T., LTD.	1925
ELFORD, H.	1924
EMERSON, HARRY B.	1926
ENGLAND, JOHN	1922
EVERHART, CALVIN C.	1925
FARMER, HOMER G.	1924
FELDRAPPE, MAX G.	1917*
FLESHEIM, S. W.	1922
FLESHER, WILLIAM	1922
FOSTER, ALEXANDER, JR.	1912*
FRANCISCO, F. LEROY	1919*
FREEMAN, PERRY J.	1923
FRIEL, FRANCIS S.	1920*
FROEHLING & ROBERTSON	1917*
FRUCHTBAUM, JACOB	1920*
GIANT PORTLAND CEMENT CO.	1914*
GIBSON, W. HERBERT	1926
GILKEY, H. J.	1924
GILL, GRAYSON	1923
GILMAN, CHARLES	1916*
GLENS FALLS PORTLAND CEMENT CO.	1908*
GOLDBECK, A. T.	1926
GONNERMAN, H. F.	1918*
GRAY, HOWARD A.	1921*
HADLEY, HOMER M.	1924
HAMILTON, CHAS. T.	1919*
HAMMILL, HAROLD B.	1920*
HANLY, HUNTER W.	1922
HATT, WILLIAM K.	1910*
HAYDEN, ROBERT	1926
HERCULES CEMENT CORP.	1924
HIRSCHBERG, WALTER P.	1920*
HOLLISTER, S. C.	1917*
HONIG, B. A.	1921*
HORN, A. E.	1924
HOWE, HARRY N.	1910*
HUBBARD, FRED	1926
HURLBURT, R. W.	1920*
HYDRO-ELECTRIC POWER COM. OF ONT.	1917*
INDUSTRIAL ENGINEERING CO.	1922
INLAND STEEL CO.	1913*
IRELAND, C. E.	1914*
IRWIN, O. W.	1916*
JACKSON, F. H.	1924
JAMES, WESLEY H.	1926
JELLICK, JAY E.	1924
JOHNSON, ROBERT C.	1925
JOHNSTON, ROBERT S.	1921*
KALINKA, JOHN E.	1925
KELLY, J. W.	1926
KLEIN, WILLIAM H.	1924
KNEESHAW, FREDERICK P.	1925
KOERNER, CARL A.	1924
KOSMOS PORTLAND CEMENT CO.	1916*
LACLEDE STEEL CO.	1921*
LANDER, R. S.	1914*
LAWRENCE PORTLAND CEMENT CO.	1916*
LEFFLER, RALPH R.	1920*
LEHIGH PORTLAND CEMENT CO.	1924
LEHIGH PORTLAND CEMENT CO.	1924
LEHIGH PORTLAND CEMENT CO.	1924
LEHIGH PORTLAND CEMENT CO.	1905†
LEVISON, ARTHUR A.	1925
LOCKE, CLYDE E.	1922
LOCK JOINT PIPE CO.	1925
LONE STAR CEMENT CORP.	1923
LORD, ARTHUR R.	1916*
LOUISVILLE CEMENT CO.	1924
LYSE, INGE	1926

*ACI Member for 30 years or more.
†ACI Charter Member, National Association of Cement Users organized in 1905.

Continued on p. 19

The Tenth of a Series in the interest of more efficient use of steel . . . . a vital American resource

LACLEDE *Multi-Rib* Reinforcing Bars

ROUND BARS in all sizes . . . to meet latest ASTM A305 Specifications . . . conveniently marked to make your specification job easier. And, of course, improved Laclede **deformations mean** uniform reinforcement strength, maximum anchorage — plus a more efficient use of steel by eliminating hooks and shortening embedment lengths.

TABLE OF ASTM A305 SPECIFICATIONS

		NOMINAL DIMENSIONS ROUND SECTIONS			REQUIREMENTS OF DEFORMATIONS		
Bar-No. *	Unit Wt. Lbs./Ft.	Diameter-Inches Decimal	Cross Sectional Area Sq. Inches	Perimeter	Max. Avg. Spacing In.	Min. Height Inches	Max. Gap. Inches °
2†	0.167	0.250	0.05	0.785	.. ..	.. .	.. ..
3	0.376	0.375	0.11	1.178	0.262	0.015	0.143
4	0.668	0.500	0.20	1.571	0.350	0.020	0.191
5	1.043	0.625	0.31	1.963	0.437	0.028	0.239
6	1.502	0.750	0.44	2.356	0.526	0.038	0.286
7	2.044	0.875	0.60	2.749	0.612	0.044	0.334
8	2.670	1.000	0.79	3.142	0.700	0.050	0.383
9‡	3.400	1.128	1.00	3.544	0.790	0.056	0.431
10‡	4.303	1.270	1.27	3.990	0.889	0.064	0.487
11‡	5.313	1.410	1.56	4.430	0.987	0.071	0.540

*Bar numbers are based on the number of ⅛ inches in the nominal diameter of the section.
†Bar number 2 in plain rounds only.
‡Bars numbered 9-10-11 correspond to former 1" sq., 1⅛" sq., and 1¼" sq. sizes, and are equivalent to those former standard bar sizes in weights and nominal cross-sectional areas.
°Chord of 12 ½% of Nom. Perimeter.

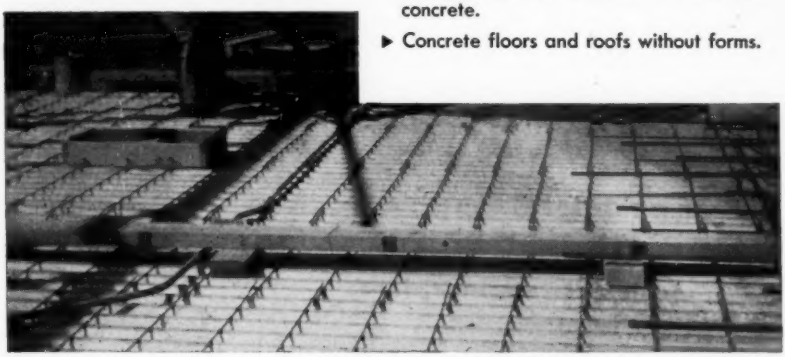
"For the *BEST* in Reinforcing Steel—Specify Laclede"

LACLEDE
Producers of Construction Steel

LACLEDE STEEL COMPANY
St. Louis, Mo.

COFAR

- ▶ Reinforced concrete without forms.
- ▶ High-strength, deep-corrugated steel manufactured with welded, closely spaced transverse wires (T-wires).
- ▶ Positive reinforcement permanently anchored to and combined with structural concrete.
- ▶ Concrete floors and roofs without forms.



STRONG

Guaranteed yield point (.1% offset) over 80,000 psi effective as reinforcement, balanced design, up to full tensile strength over 100,000 psi, independent of form stress.

VERSATILE

Minimum 24 ga. —(A_s .35 sq. in. per foot width); maximum 18 ga. —(A_s .73 sq. in. per foot width); to meet full range concrete slab design. COFAR reinforcement satisfies all normal concrete slab design.

Cold-drawn, high-strength T-wires not over 6″ c.c. are welded to the deep-corrugated steel, in manufacture, and constitute temperature reinforcement in the slab, mechanical anchorage and positive shear transfer from concrete to steel.

PERMANENT

COFAR sheets hot-dip galvanized, with specification heavy galvanizing, provide complete permanence for normal interior exposures. Concrete and steel permanently bonded by chemical union of calcium zincate for all normal loads. Fire resistance for any exposure by lightweight, modern ceiling plaster or fibre protection.

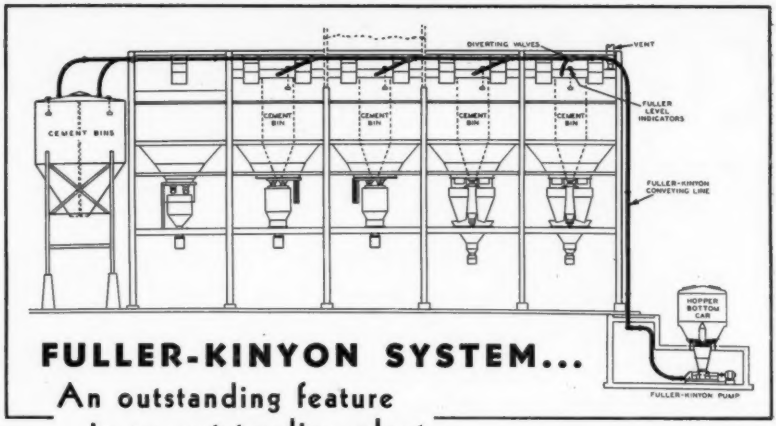
ECONOMICAL

COFAR ***is concrete reinforcement,*** steel weight substantially equal to conventional reinforcement; COFAR ***also is the concrete form.*** Form sheeting as well as concrete runways are eliminated. COFAR comes detailed and cut to fit each job. Important material, labor, and time savings are made.

Advice for design engineers, review and estimates. COFAR is available — Granco Steel Products Company.

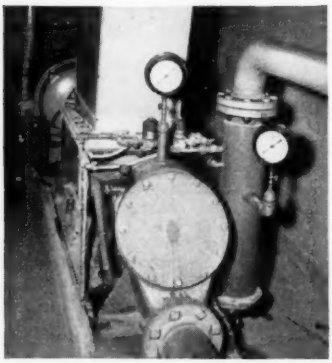
GRANCO STEEL PRODUCTS CO.
(Subsidiary of **GRANITE CITY STEEL CO.***)*
GRANITE CITY, ILLINOIS

SEND FOR AIA FILE NO. 4E4



FULLER-KINYON SYSTEM...

An outstanding feature
in an outstanding plant

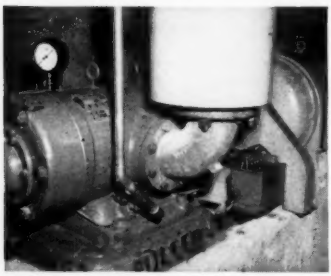


Fuller-Kinyon Pump in pit underneath tracks.

One of the ready-mixed concrete plants put into operation during 1950, Iron City Sand and Gravel Company, Pittsburgh, Penna. uses a Fuller-Kinyon System for handling bulk cement. It is said the storage and conveying system is one of the important features of this plant.

Cement, received by rail or truck, is discharged to a Fuller-Kinyon Pump, which conveys to one storage bin of 800 barrel capacity of two compartments, and four 200 barrel compartments built into the center of each of the four-compartment aggregate bins.

The system delivers to all bins through diverting valves in the conveying line, which are operated from a walkway above the bins. Automatic bin controls sound an alarm and shut down the pump whenever a bin is full. The operator can start and stop the pump from a central control panelboard.



Fuller Rotary Compressor which furnishes air for conveying—capacity 482 c.f.m., 20-lb. pressure.

A Fuller-Kinyon System may be readily installed without interrupting production or interfering with existing structures or equipment. Why not have us survey your conveying systems and make a recommendation for the betterment of your operations. There's no obligation, of course.

FULLER COMPANY, Catasauqua, Pennsylvania
Chicago 3—120 So. LaSalle St.
San Francisco 4—420 Chancery Bldg.

Fuller P-132

ACI Members for 25 Years or More

Continued from p. 15

Name	Date
Maine, University of	1926
Marquette Cement Mfg. Co.	1924
Masters, Frank M.	1926
McDaniel, Allen B.	1915*
McMillan, F. R.	1916*
Mead, F. J.	1923
Medusa Portland Cement Co.	1916*
Menefee, F. N.	1925
Meyer, Adolf A.	1926
Meyer, George W.	1924
Milburn Lime & Cement Co.	1924
Minshall, R. E.	1924
Mitchell, Nolan B.	1921*
Morris, Clyde T.	1920*
Nazareth Cement Co.	1916*
Newlin, James C.	1922
Noonan, W. H.	1920*
North Jersey Quarry Co.	1921*
Northwestern States P. C. Co.	1908*
Oehmann, John W.	1921*
Olsen, Edward	1923
Parsons, Douglas E.	1926
Patzig, Monroe L.	1924
Peabody, Dean, Jr.	1914*
Pearse, Langdon	1914*
Pennsylvania State Highway Dept.	1911*
Pittsburgh Testing Laboratory	1912*
Platwit, Eric	1921*
Polaris Concrete Products Co.	1923
Pollak, Julian A.	1923
Puerto Rico College of A. & M.	1926
Puerto Rico Dept. of the Interior	1924
Quebec Dept. of Public Works	1921*
Quebec Dept. of Roads	1920*
Rail Steel Bar Assn.	1926
Randall, Frank A.	1918*
Raymond Concrete Pile Co.	1916*
Rebolledo, Miguel	1924
Rishel, John A.	1926
Robinson, G. G.	1925
San Antonio Portland Cement Co.	1926
Saum, Irving R.	1922
Scherer, Francis R.	1926
Schnarr, Wilfrid	1925
Scholer, C. H.	1924
Scott, J. Russell	1921*
Selmer, F.	1926
Shank, J. R.	1922
Sherman, Ralph A.	1923
Shields, Harry C.	1926
Singleton-Green, John	1921*
Smidth & Co., F. L.	1925
Smith, W. H.	1922
Societe Anonyme des Chaux et Ciments de Laboratoire Central	1925
Solvay Process Div., Allied Chemical & Dye Corp.	1924
Southwestern Portland Cement Co.	1926
Spiker, Wm. C.	1918*
Stepanian, S.	1925
Stone & Webster Engineering Corp.	1920*
Straub, Lorenz G.	1926
Sutherland, Hale	1916*
Thompson & Lichtner Co., Inc.	1925
Totti, Etienne	1926
Turner Construction Co.	1919*
Turner Construction Co.	1920*
Turner, Henry C.	1905†
Tuthill, Lewis H.	1926
Universal Atlas Cement Co.	1914*
Upson, Maxwell M.	1912*
Urquhart, L. C.	1923
Van Buren, Maurice P.	1926
Voss, Walter C.	1917*
Walker, Frank R.	1923
Walker, Stanton	1921*
Wallace, William K.	1915*
Walter, Ernest	1920*
Warner Co.	1910*
Watt, Archie G.	1925
Way, William F.	1921*
Whipple, Harvey	1919*
Whitehall Cement Mfg. Co.	1908*
Whitney, Charles S.	1920*
Wickersham, John D.	1919*
Wigton, C. Benson	1913*
Wilk, Benjamin	1925
Williams, Charles H.	1922
Williams, G. M.	1919*
Wilson, Francis W.	1925
Wilson's New Zealand P. C. Co.	1910*
Withey, Morton O.	1921*
Witt, J. C.	1924
Witte, Herman C.	1926
Wu, Pond S.	1921*
Young, Roderick B.	1917*
Young, Roy N.	1921*

*ACI Member for 30 years or more.

†ACI Charter Member, National Association of Cement Users organized in 1905.

Bylaws revision ratified

The revision of ACI Bylaws proposed and adopted at the 47th annual convention in San Francisco, February 1951 (see January NEWS LETTER for complete text) was ratified by letter ballot of the Institute membership canvassed June 26, 1951.

The new Bylaws changed Individual dues from $12.00 to $15.00, Contributing Member dues from $50 to $100 and Corporation Member dues from $25 to $50. Individual Member dues for those not residing in North America* were raised from $10 to $12. There was no change in Student or Junior Member dues.

Sections were changed in Article I to define membership grades more specifically, and require only one sponsor for a new member. Duties of officers were clarified in Article II.

Complete Bylaws may be found in the 1951 Directory.

Building code gets ASA number

The recently revised "Building Code Requirements for Reinforced Concrete (ACI 318-51)" has been approved as an American Standard by the American Standards Assn. and given ASA Designation A89.1-1951.

*Formerly this group excluded United States, its possessions and Canada only. It has now been changed to North America which includes all countries and territories in Central America and the West Indies. Authorities: *Commercial Atlas of America*, Rand McNally & Co., Chicago; and *Survey Atlas of the World*, The Times, London, England.

Positions and Projects — ACI Members

Appointments to ACI Technical Committees since 1951 Directory listings closed

Committee 208, Bond Stress

W. H. Jacobs
Secretary, Rail Steel Bar Assn.
38 S. Dearborn
Chicago 3, Ill.

Committee 212, Admixtures

Charles E. Wuerpel
Technical Director
Marquette Cement Manufacturing Co.
20 N. Wacker Drive
Chicago 6, Ill.

Committee 314, Rigid Frame Bridges

George J. Kerekes
149 Lunado Way
San Francisco, Calif.

Committee 315, Detailing Reinforced Concrete Structures

Lynn H. Hench
Chief, Structural Design Section
Engineering Div., Military Construction
Office, Chief of Engineers
Washington 25, D. C.

John M. Kerr*
Chief, Structural Division
Technical Service,
Veterans Administration
Washington 25, D. C.

Charles J. Kuhn*
Kuhn Construction Co.
Kanawha Valley Bank Bldg.
Charleston 1, West Va.

Committee 318, Standard Building Code

A. C. Weber
Director of Research and Sales Engineering
Laclede Steel Co.
Arcade Bldg.
St. Louis 1, Mo.

Committee 611, Inspection of Concrete

David Ehrenpreis
2214 63rd St.
Brooklyn, N. Y.

*Non-Member ACI

Committee 711, Precast Floor Systems for Houses

Ralph W. Adams
Consulting Engineer
32 N. Washington St.
Ypsilanti, Michigan

ASA Sectional Committee (A37) Road and Paving Materials

Frank H. Jackson has been appointed to succeed R. W. Crum, deceased, as ACI representative.

Hollister heads ASEE

At the annual meeting of the American Society for Engineering Education, ACI past-president S. C. Hollister was elected president of ASEE. Hollister is Dean of Engineering at Cornell University and occupies an important place among engineering educators in this country. Recent work in this connection has been the compilation of information showing actual needs for engineering graduates in government and industry in the United States.

Klein and Walker receive merit awards

William H. Klein, vice-president and operating manager, Lawrence Portland Cement Co., Northampton, Pa. and Stanton Walker, director of engineering and research, National Sand and Gravel Assn. and National Ready Mixed Concrete Assn., Washington, D. C., were among the ten who received ASTM Awards of Merit at its 54th annual meeting in June.

The award is given to those who have rendered outstanding service to ASTM, particularly in technical committee work. Mr. Klein was cited for his outstanding activity in Committee C-1 on Cement; Mr. Walker, for outstanding work in Committee C-9 on Concrete and Concrete Aggregates, of which he was secretary for 15 years.

Mr. Walker was also elected to ASTM's Board of Directors for a 3-year term. Both men also have been active in ACI Technical and administrative committee work. Mr. Klein is author of several JOURNAL papers and has served on ACI's Publications Committee and Board of Direction. Mr. Walker, a past president of the Institute, is a prolific

writer, besides serving on the Board of Direction and various other committees.

ASTM elects honorary members

In recognition of their eminence in technical work in the field of engineering materials and meritorious service to ASTM, the society honored A. T. Goldbeck, H. H. Morgan and Frank E. Richart by awarding them certificates of Honorary Membership at its annual meeting in June.

Mr. Goldbeck, engineering director, National Crushed Stone Assn., Washington, D. C., is now vice-president of ACI, and active in technical and administrative work. Mr. Morgan, vice-president and general manager, Robert W. Hunt Co., Chicago, was president of ASTM, 1939-40, and has been a leader in the testing of cement, railroad materials and concrete reinforcement.

Professor Richart, who died in July, was for many years research professor of engineering materials, University of Illinois, Urbana, and one of the country's leading authorities on concrete and reinforced concrete. A past president of ACI, he was the recipient of numerous awards, among them the ACI Wason Medal for noteworthy research, with T. A. Olson, in 1937 and the first Alfred E. Lindau Award in 1949 "for his years of labor toward the improvement of reinforced concrete design practice."

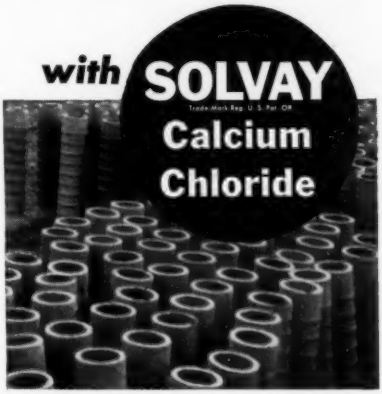
Building research congress

The Building Research Congress being held September 11-20, 1951 during the last month of the Festival of Britain will include papers by several prominent ACI members. A general review of the influence of modern research on structural design will be presented by Prof. Georg Wastlund of Sweden, Dr. A. R. Collins, Great Britain, will discuss research and development of prestressed concrete, Robert F. Blanks will talk on concrete quality control and H. F. Gonnerman will discuss the durability of concrete structures.

Morgan returns to Army

Newlin D. Morgan, Jr., formerly with the CE Department, University of Wyoming, and an outstanding ACI member-getter, has been recalled to active duty as a Major in the Corps of Engineers. He is presently organizing an Engineer ROTC unit at the University of Detroit.

DOUBLE YOUR OUTPUT



With the addition of Solvay Calcium Chloride, good concrete products can be made better, faster, at lower cost. Solvay Calcium Chloride helps improve the quality of your products, enables you to strip forms in half the time, gives protection against low temperatures and offers many other advantages.

FREE BOOK AVAILABLE

For complete details, send for free copy of "How to Get Better Concrete Products at Lower Cost." Contains complete information on the use of Solvay Calcium Chloride in cast concrete products. Fill in and mail coupon today.

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Please rush my free copy of "How to Get Better Concrete Products at Lower Cost."

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Company.....
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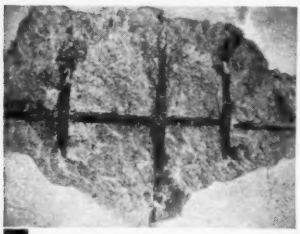
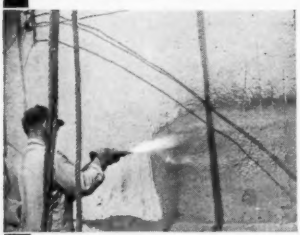
JC-9



Typical example of spalling. Note corrosion of reinforcing rods exposed by disintegrating concrete.



Reinforced with meshing, the area is restored with gun-applied RESTO-CRETE* by Western Waterproofing Co.

FOR *Lasting* CONCRETE RESTORATION

Specify GUN-APPLIED

RESTO-CRETE*

by WESTERN WATERPROOFING CO.

Sound engineering methods, finest materials, trained technicians and over 35 years' experience assure the job will be done *right* when you specify Western. All work done under contract, fully insured, performance guaranteed. *T. M. Reg.

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Branch Offices and Resident Engineers in Principal Cities

Murphy gets ASEE award

Glenn Murphy, professor of theoretical and applied mechanics and senior engineer for the Iowa State College Institute for Atomic Research, was named recipient of the American Society for Engineering Education George Westinghouse Award at its annual meeting in June. The award was made on the basis of Dr. Murphy's pre-eminence as a teacher of engineering.

An abstractor for a number of years for the "Current Reviews" section of the JOURNAL, he is the author of numerous books, papers and bulletins, besides being active in vasious technical societies.

Dr. Murphy received BS and MS degrees from the University of Colorado in 1929 and 1930, respectively, and CE degree in 1937. After a 2-year assistantship at the University of Illinois, he received his second MS degree.

He joined the staff of the theoretical and applied mechanics department at Iowa State in 1932 and received a doctor's degree in 1935. He was promoted to full professorship in 1941. He has been senior engineer in the Institute for Atomic Research since 1949, in addition to his teaching duties.

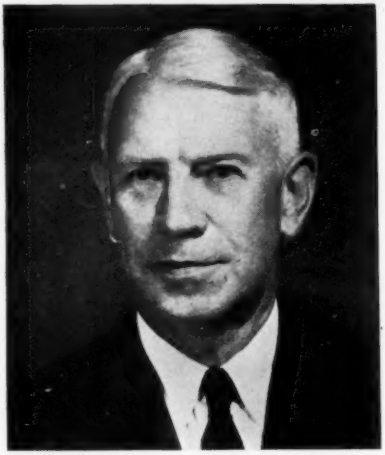
Steinman awarded Norman Medal

D. B. Steinman has been awarded the Norman Medal of the American Society of Civil Engineers for his paper entitled "Aerodynamic Theory of Bridge Oscillations." The award will be formally presented at the ASCE annual convention in New York, October 24. The Norman Medal is the highest award conferred by the Society for published papers of recognized merit and constituting original contributions to engineering.

This is the fourth ASCE award to Dr. Steinman, and the second time he has won the Norman Medal. Other awards are the J. James R. Croes Medal (1919) and the Thomas Fitch Rowland Prize (1929).

Phillips retires

Thomas L. Phillips, chief engineer, Western Pacific Railroad Co., San Francisco, has retired after 44 years service. Frank R. Woolford succeeds Mr. Phillips.



Frank E. Richart

Professor Frank E. Richart, president of ACI in 1939 and member of the Board of Direction from 1934 to 1945, died July 16 at his home in Urbana, Illinois. Because of ill health he had been on leave of absence from the University of Illinois since February, 1950.

He received the ACI Leonard C. Wason medal in 1938, the Lindau medal in 1949, and honorary membership in ACI and ASTM in 1951. He had recently been first vice-president of ASTM after service on its executive committee and its board of directors.

Other organizations in which he was active, and in several of which he held offices, are Western Society of Engineers, ASCE, Society for Experimental Stress Analysis, Sigma Xi, Phi Kappa Phi, and Tau Beta Pi. He represented the Western Society of Engineers on the Highway Research Board.

Professor Richart had been in charge of concrete research in the University of Illinois college of engineering and the engineering experiment station since 1926, supervising or conducting intensive studies of highway bridges, bridge parts, and bridge slabs. His studies of the design and use of reinforced concrete had made him internationally known.

He received his BS in civil engineering from the University of Illinois in 1914, his MS there in 1915, and the professional degree of Civil Engineer in 1922.

From 1913 to 1917 he was engaged in railway maintenance, valuation field work, reinforced concrete building detailing and designing; testing engineer, Concrete Ship Section, Emergency Fleet Corp., 1917 and 1918. Since 1916 he had been in the Department of Theoretical and Applied Mechanics at the University of Illinois, becoming a full professor in 1931. He was the author of many bulletins and articles on indeterminate structures, proportioning of concrete by mortar-voids method, tests of reinforced concrete beams, columns, slabs, walls, footings, rigid frames, flat slab floors, reinforced masonry, and other subjects. He was chairman of the Institute's Committee on Reinforced Concrete Column Investigation and was in personal charge of that portion of the investigation carried on at the University of Illinois. Data derived from this program became the basis for the column chapter of the ACI Building Code.

He was also noted as a director of the University of Illinois Athletic Assn. from 1936 to 1941 and as the University's faculty representative on the Big Ten board since 1942.

Dean Peabody, Jr.

Dean Peabody, Jr., professor of architecture in the Harvard University Graduate School of Design and an ACI member since 1914, died at Hubbardston, Mass., August 7.

Professor Peabody was a member of ACI Committee 318 and before joining the staff of Harvard University in 1948 was associate professor of structural design at Massachusetts Institute of Technology. He received his SB degree from MIT in 1910 and the SM degree in 1938.

During the summers of 1911 to 1916 he worked in the field or design offices of the Aberthaw Construction Co., New England Concrete Construction Co., Lockwood, Greene and Co., and Stone and Webster. He was the author of *Reinforced Concrete Structures* and contributed to ACI technical literature.

He was a member of Chi Epsilon, Sigma

Xi, ASCE, ASTM, AAAS and the Boston Society of Civil Engineers.

Carlton S. Wicker

Carlton S. Wicker, 62, a civil engineer, died May 17 in Buffalo, N. Y. He was sales manager of the Buffalo Slag Co. for many years. He was a graduate of Dartmouth College.

ASTM honors USBR authors

Richard C. Mielenz, Leslie P. Witte and Omar J. Glantz, all with the Bureau of Reclamation, Denver, were honored with the Sanford E. Thompson Award at the 1951 ASTM annual meeting in June. The award, given for a paper of outstanding merit on concrete and concrete aggregates, was presented the authors for their paper "Effect of Calcination on Natural Pozzolans."

Mr. Mielenz, with Kenneth T. Greene and Elton J. Benton, received ACI's Wason Medal for noteworthy research in 1947 for the work reported in their paper, "Chemical Test for Reactivity of Aggregates with Cement Alkalies; Chemical Processes in Cement-Aggregate Reaction."

Britton retires

George C. Britton, active for 24 years in Portland Cement Assn. work, has retired. He joined the association staff in 1927 as field engineer in Pennsylvania and in 1933 was appointed district engineer at Philadelphia in charge of services in Pennsylvania, Delaware and Maryland. He is succeeded by Robert M. Reindollar, Jr., formerly assistant district engineer. Mr. Britton will continue on a part-time consulting basis.

Fago transferred

Victor S. Fago, formerly hydraulic engineer with the San Diego section of the International Boundary and Water Commission, is now Chief of Engineering Branch, Corps of Engineers, Rhine Military Post, Germany.

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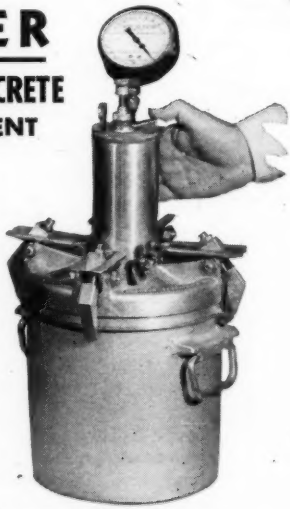

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113 Individual, 6 Corporation, 27 Junior and 25 Student applicants were accepted during May, June and July bringing the Membership total to an all-time high of 5347 as of August 1.

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ALLEN, RONALD, Somerset, England (Products Mfgr.)
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BIRKELAND, HALVARD W., San Francisco, Calif. (Asst. Chf. Design Engr., International Engrg. Co.)
BRETT, JOHN E., Hoboken, N. J. (Asst. Prof. & Chairman, Stevens Inst. of Tech.)
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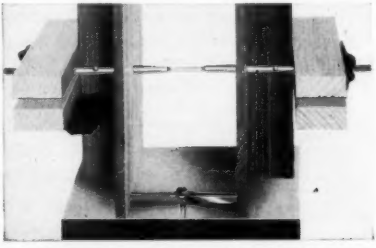

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Who's Who

Continued from p. 12

istry at the University of Toledo. Previous work of these fellowships was reported in the June 1949 JOURNAL.

From 1930 to 1944 Mr. Kalousek engaged in research on the physical and chemical properties of cements and concrete at the National Bureau of Standards. These studies have been reported in 15 publications.

Mr. Kalousek graduated from the University of Idaho in 1930 with a BS in Ch. E. MS and PhD degress in chemistry were received in 1936 and 1941, respectively, from the University of Maryland.

Milton Adams, following a 4-year service in the U. S. Army during his college career, graduated *magna cum laude* from the University of Toledo with a BE degree in 1948. Appointed a fellow in silicate chemistry he received his MS there in 1949. He is continuing his studies toward a PhD at Massachusetts Institute of Technology.

Emergency building code

A proposed model emergency building code ordinance which will permit local building code officials to authorize use of alternate materials and techniques during the present emergency has been prepared by the Housing and Home Finance Agency. The agency's Advisory Committee on Administration of Local Building Codes recently approved the ordinance. Copies will be available in the near future.

Sandwich-type building walls

A new building for the Pittsburgh Corning Corp., Port Allegany, Pa., has prefabricated cellular glass insulated sandwich wall units consisting of two veneers of concrete with a core of cellular glass. The roof is insulated with the same material.

The sandwich type walls are being cast on the flat on the building site and erected in various sizes; generally in 12 x 4-ft units. This construction method is expected to reduce cost considerably and to make earlier completion possible.

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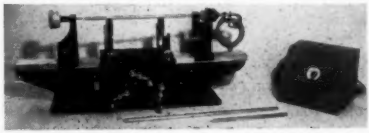


Length-change comparator for concrete specimens

The Waterways Experiment Station, Corps of Engineers, U. S. Army, has developed a modified length-change comparator (see above) for use in studies of concrete specimens. Recently incorporated modifications include the substitution of an electrical contact indicator for the dial gage formerly used as one of the measuring devices. Two comparators of modified design are now in use in Corps of Engineers laboratories. These instruments permit the detection of changes of as little as 0.0001 in. in the length of a concrete specimen and can be used with either cylindrical or prismatic specimens of any dimensions up to 10 in. in diameter and 24 in. long. The comparator was originally developed to meet the need for an instrument capable of making such measurements on specimens that could not be accommodated on a vertical comparator.

French study here

Under the technical assistance program of the Economic Cooperation Administration a 12-man team of technologists from the

French cement industry recently completed a study of American manufacturing methods. Among those contributing to the success of their visit were Universal Atlas Cement Co., Warner Co., Allentown Portland Cement Co., Alpha Portland Cement Co., Penn-Dixie Cement Corp., Hercules Cement Corp., Missouri Portland Cement Co., Lone Star Cement Co. and the Portland Cement Assn.

Newsprint warehouse

A 200,000 sq ft, two-story newsprint terminal is being completed by the Turner Construction Co. for the New York *News* in Brooklyn. The building, designed by Fay, Spofford and Thorndike of Boston, is of reinforced concrete construction. Ground floor and all exterior walls and columns are supported by concrete piles. Normal working paper storage capacity is 32,000 tons; maximum is 39,000 tons.

Pozzolan defined

Committee C-1 on Cement, American Society of Testing Materials, has recommended the following tentative definition of "Pozzolan" for acceptance:

"The term pozzolan is used to designate a siliceous or aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties."

The committee also recommended revisions in "Methods of Test for Time of Setting of Hydraulic Cement," "Standard Method of Test for Fineness of Portland Cement by the Turbidimeter," and "Standard Methods of Chemical Analysis of Portland Cement."

Fellowship in housing

The Albert Farwell Bemis Foundation of Massachusetts Institute of Technology, Cambridge, Mass., has created an annual fellowship in housing, granting up to $2,500 to a qualified candidate submitting a program for a year of graduate study in any one of a wide range of fields related to housing. The program may or may not lead to a graduate degree, and it may involve courses in any one or combination of a number of departments at the Institute.

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Safety pays

"Safety Pays the Smaller Business," a 24-page booklet issued by the National Safety Council, cites the problem of accident prevention for small business and offers solutions to the problem. A nine-point program is suggested as a means to reducing accidents in the small plant.—National Safety Council, 425 N. Michigan Ave., Chicago 11, Ill.

Masonry drill bit

A removable "Cyclo-Center" is said to eliminate the necessity of using a wood template to start drill holes in concrete, tile or masonry. The special center is located in the drill bit and removed after the hole is spotted. The "Cyclo-Core" carbide tipped drill bits are available in ¾-in. diameters and larger.—New England Carbide Tool Co., Cambridge 39, Mass.

Form clamp

The principal feature of a new "Hex-Lock" form clamp is the combining into a single unit of an adjustable hex-lock floating nut and bracket. The manufacturer says that this combination permits the turning of metal on metal, reducing friction since the nut units are not rotated directly against the wood walers.—Williams Form Engineering Corp., 1501 Madison Ave., S.E., Grand Rapids 7, Mich.

Gravel plant

A single pass gravel plant is described in Bulletin CP-1 from Diamond Iron Works, Inc. Three plants, with capacity range from 20 to 65 tons per hours, are described and specifications listed.—Diamond Iron Works, Inc., Minneapolis 11, Minn.

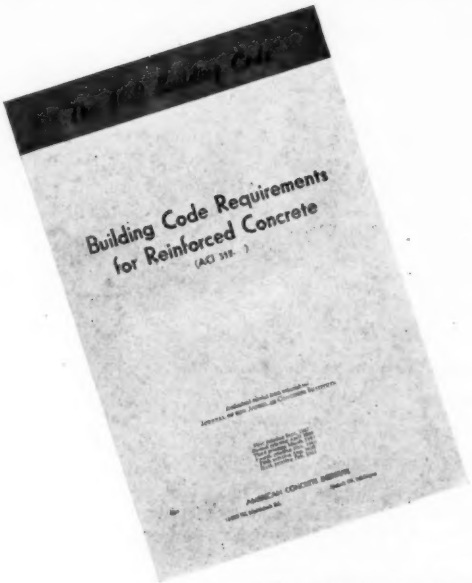
Roof fill

Lightweight insulating concrete roof fill is described in a new bulletin issued by Great Lakes Carbon Corp. Properties, mix design and typical specifications for Permalite concrete roof fill are presented.—Great Lakes Carbon Corp., Building Products Division, 18 East 48th St., New York 17, N. Y.

Masonry cutting blade

A new masonry-cutting blade developed to reduce blade breakage is manufactured from layers of glass fiber cloth impregnated with resins and silicon carbide. This material is then pressed together under hydraulic pressure and processed in kilns. The manufacturer states that the Clipper "B-R" blade is most effective in the softer range of materials such as limestone, sandstone and lightweight concrete products.—Clipper Manufacturing Co., Dept. AC, 2800 Warwick, Kansas City 8, Mo.

ACI Building Code



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New ACI Code allows for properties of A305 bars
RESULT—steel savings in reinforced concrete construction

The Code covers the proper design and construction of buildings of reinforced concrete. It is written in such a form that it may be incorporated verbatim or adopted by reference in a general building code.

Changes in the ACI Code decrease the allowable bond stress in plain bars (including old-type deformed bars) and increase allowable bond stresses for new-type bars. According to the new provisions, all plain bars must be hooked, but the new reinforcing bars develop sufficient anchorage by bond to correspond to special anchorage under the old Code. The ACI Code was revised to give designer and builder the benefit of improved bond stresses of new bars, with its ensuing economy.

The Code also covers: mixing, placing, curing and cold weather protection of concrete; forms; cleaning, bending, placing, splicing and protection of reinforcement; construction joints; general design considerations; flexural computations; shear and diagonal tension; bond and anchorage; allowable stresses; flat slabs; columns and walls; and footings.

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DISCUSSION

Discussion closes January 1, 1952

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Earthquake Resistant Design Considerations—R. R. Martel

Comprehensive Numerical Method for the Analysis of Earthquake Resistant Structures—Charles S. Whitney, Boyd G. Anderson and Mario G. Salvadori

Multistory Buildings Designed to Resist Earthquakes—John J. Gould

Lightweight Concrete for Lower Construction Costs—J. A. Murlin

Designing for Continuity in Prestressed Concrete Structures—Alfred L. Parme and George H. Paris

Pumice—Lightweight Aggregate—Leslie I. Neher

Hydration Products Formed in Cement Pastes at 25 to 175 C—George L. Kalousek and Milton Adams

Advances in Precast Floor Systems*

By F. N. MENEFEE†

SYNOPSIS

Reinforced concrete floors and roofs for office and school buildings, apartments, and similarly loaded structures are being made of precast blocks assembled to form a "plank" or "slab," precast joists with a precast filler placed between, and 4-ft wide precast slabs supported by two integrally precast T-joists.

The high cost of carpentry has brought about a successful attempt to eliminate most of the previously required formwork. The trend is toward units made of high strength, lightweight aggregate under closely controlled factory conditions.

In one of the plank or slab types herein reported the steel is given a stress of approximately 18,000 psi, but it is believed that most of this prestress is lost through shrinkage and plastic flow by the time actual working loads are imposed.

Special shape block forming and grinding equipment produces blocks to be assembled into floor plank or slabs held in compressed relationship by threaded steel reinforcing rods. Precast inverted T-joist forming machinery makes joists to be used with a special shaped hollow filler block to make a floor which may be laid without form work.

Tests show that for the most part standard reinforced concrete theory is applicable to these new floor systems. Where there is doubt as to the theory covering the new types, tests are being made under Sec. 103 (a) ACI Building Code (318-51) and Minimum Standard Requirements for Precast Floor Units ACI 711-46, resulting in widespread approval.

INTRODUCTION

Precast reinforced concrete floor systems made from the I-beam and hollow-core joists such as described in Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46) are still popular. However, other systems which are adding to the popularity of precast construction have been recently developed.

Those that are, for the time being, attracting the most attention are (a) Dox Floor and Roof System, (b) F and A, (c) JoisTile, and (d) Strestcrete. In addition to the foregoing there is the less well known integrally precast slab and T-joist known in Florida as the "Farhan" slab.

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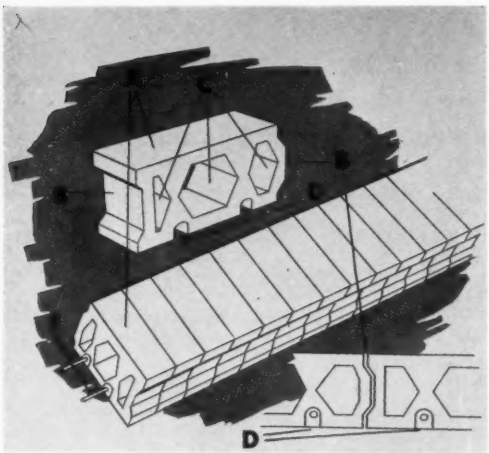


Fig. 1—Dox floor and roof system. A—Modular surfaces. B—Tongue and groove interlocks beams, distributes load and aligns floor surface. C—Openings in blocks reduce weight and provide space for utilities. D—Recessed channels space and anchor reinforcement. E—Reinforcing bars in place.

DOX SYSTEM

In the Dox System the lightweight block are tongue and grooved as shown in Fig. 1; this interlocking feature automatically aligns each plank or slab with those on either side giving a level working surface.

Based on the theory for simply supported beams, 6-in. Dox block floor or roof plank may be loaded to the values in Table 1.

After being cast Dox block are tapered by grinding the faces perpendicular to core openings from top to bottom, the bottom dimension being the smaller (Fig. 1). Enough blocks to form a beam of the desired length are placed side by side bottom up and a steel rod is run through the center openings. A hydraulic device stresses the rod to bring the blocks snugly together, putting a camber in the beam. A wedge is inserted through the rod to hold the tension until the beam is cured. The slots in the beam are then wetted and filled with mortar which is vibrated to insure proper cover of the rods. The beam is cured, turned over, and eventually placed in a floor or roof.

TABLE 1—SAFE LOADS FOR DOX BEAMS

Span, ft	Live load, psf	Application
20	40	Dwelling floors, roofs. Bed and living room floors of apartments, hospitals, hotels, clubs and similar buildings.
19	50	Office buildings, other than first floor; schools with fixed seats.
16	80	Assembly halls with fixed seats, in churches and apartments. First floor corridors and dining rooms, hotels and clubs. Restaurants, schools, theaters, auditoriums, balconies, galleries, foyers, lobbies. Floors other than bed and living in hospitals, asylums; lofts and shops other than first floor.
14	100	Assembly halls and churches (movable seats); grandstands; manufacturing, mercantile and retail stores (other than first floor).
12	125	Dance halls, drill halls, first floors of lofts, shops, mercantile and office buildings and retail stores.

The beams are trucked to the building site and hoisted into place by a crane lifting on a sling clamped to the opposite ends. Since some of the normal shrinkage in the blocks takes place before they are assembled into a beam it is probable that the resid-

ual shrinkage of the concrete is less than would take place in a completely poured in place floor.

The lightweight, hollow-cast block provides a surface to which the top coat bonds and into which negative reinforcing mesh can be placed when desirable or necessary. The concrete top coat is laid when the camber is still in effect, thus increasing the thickness of the T section and may be taken into consideration in computing allowable loading. In light construction, finish flooring of terrazzo, parquet, rubber tile, linoleum, or carpeting may be put on with or without an intervening topcoat of concrete or even grout. For roofing, asphalt may be spread directly over the block.

The underside forms a flat ceiling which may or may not be plastered. Cement paint in appropriate colors directly on the block produces a pleasing textured effect.

The 65 percent core space provides insulation against heat and sound transfer as well as ducts for wiring or piping. In some cases they may be used for cold air ducts.

The floors are lighter than monolithic gravel or stone concrete, providing relatively small dead load. Such deflection as will occur merely uses up the camber. Being factory made where supervision and control is superior to on-the-job construction, the uniformity of the product is greatly enhanced.

George Mansfield, General Manager of Concrete Plank, Inc., a producer of Dox blocks, says:

The Dox block has been tested consistently at an average of 2100 psi on the net area of the unit with a modulus of elasticity approximately 1,300,000 psi.

The Dox system with 8-in block, when fabricated and containing the grout and reinforcing steel, weighs 30 psf. The minimum 1-in. concrete topping required weighs 12½ psf.

The negative moment can be taken care of in two ways: (1) Reinforcing steel placed in the topping which will probably require an increase in topping thickness. The bond between the topping and cinder plank is very strong and may be computed with the top portion of the block as a part of the T in a T section, (2) We manufacture a Dox block with grooves in the upper face similar to the grooves that receive the tension steel. Reinforcing steel can be placed in the upper grooves to connect abutting plank and thereby provide resistance to negative moment.

For cold weather floor and roof construction, the Dox system is highly regarded. Under favorable conditions 1500 sq ft of 20-ft span can be laid in an hour, providing a ceiling for the floor underneath and a deck for the floor above. Salamanders soon heat up the working space below to improve working conditions.

There are no forms to be installed or removed. The general contractor knows in advance just what the floor or roof will cost and knows that it will be competently laid and in minimum time.

Reports from contractors indicate that the over-all costs are reduced as much as 10 percent and that construction time is reduced even more so.

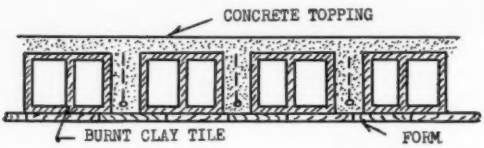


Fig. 2—Composite clay tile and concrete construction of floors

F AND A SYSTEM

The F and A* system is an outgrowth of the old floor construction method in which hollow burnt clay building tile were laid end to end in rows from 4 to 6 in. apart on a wood framework or platform. Reinforcing steel was laid on chairs between the rows and concrete placed over the tile and in the spaces between them. The 2 or 2½ in. of concrete floor topping, with the concrete and steel between the tile formed a series of T-beams each separated by a tile (Fig. 2).

Subsequently the precast I-beam joist was developed, in depths from 8 to 14 in. (See ACI 711-46). With the joist in position and the concrete placed so as to imbed the joist from ½ to ¾ in., a T-beam was formed making a lightweight, fully reinforced floor.

The F and A system, sometimes called the filler block floor system, employs an inverted T-beam joist. The flanges of the inverted T form a shelf on which the filler blocks rest. The concrete floor topping is then placed over the filler blocks and top of the joist to which it bonds, forming a T-beam (Fig. 3).

The joists are placed 24 or 28 in. on centers. Their own dead load deflection is taken out by temporary shoring at the center of the span until the filler block are in place and the concrete topping has set. With final set of the 3000 psi concrete topping, the resulting T-beam action is computable as for floor loads, by standard reinforced concrete design, in which the top of the filler block is included where it comes within the dimension *Kd*. While not vouching for all of the loadings given in Table 2, joist loadings in a few cases have been checked and found to be correct. *Concrete Products*, October 1949, reports a test of a 6-ft wide panel by the Pittsburgh Testing Laboratory, in which a load of one and one-half times design load, plus one half of the dead load was placed with a resulting deflection of 1/534 of the clear span of 20 ft 4 in. After sustaining the load for 24 hours, the recovery was 71.7 percent upon removal.

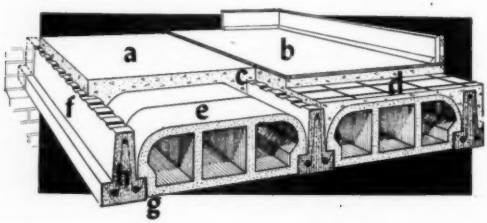


Fig. 3—F and A system. a—3000 psi concrete topping, 2-in. minimum thickness. b—Finish floor surface. c—3 x ¼-in. keys, 6 in. on centers. d—Mesh or bar reinforcement. e—8-in. filler block, 21 in. x 6, 8 or 10 in. f—joist, 6, 8 or 10 in. and 24 in. on centers. g—Ceiling finish. h—Reinforcing bars and stirrups

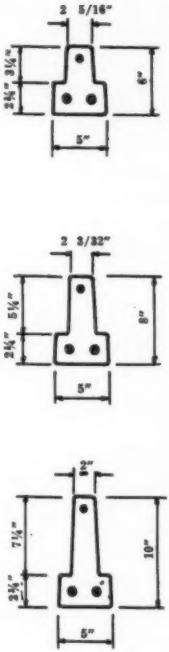
*Formigli Bros., Inc., Philadelphia; and Arnold Stone Co., Greensboro, N. C.

TABLE 2—SAFE SUPERIMPOSED LOADS—F AND A FLOOR SYSTEM

Based on: 28 in. spacing of 5-in. wide joists, 25 in. wide filler blocks. f_c = 1350, f_s = 20,000 psi, n = 10. When selecting floor joists; add to the live load weight of floor finish, ceiling and distributed partition loads.

SIZE OF JOIST	WL² / 8		SAFE SUPERIMPOSED LOAD LBS. P.S.F. ON FLOOR AREA — SPAN IN FEET																						
	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
623	143	91	57	33																					
624	154	146	99	65	43	26																			
644	304	196	130	99	69	47	31																		
645	389	252	195	107	99	69	56	39																	
655	490	346	211	151	141	107	67	56	45	32															
656	617	437	323	243	163	136	111	86	69	61	34														
666	677	531	394	300	232	183	139	111	91	72	57	30	26												
667		567	476	364	285	235	181	145	99	90	79	58	50	21	17										
677			490	439	337	270	217	177	145	116	99	73	65	53	41										
678					381	319	259	213	176	146	122	102	84	60	53										
688						343	304	249	208	174	136	122	104	76	73										
689							310	279	243	204	173	147	126	106	91										
699									252	233	201	171	147	125	109										
823	201	131	87	56	33	17																			
834	251	202	140	99	69	46	29																		
844	301	270	171	141	102	75	52	36	23																
845	518	343	267	198	129	114	85	63	47	33															
855	681	463	291	251	195	152	97	85	66	54	40	28													
856		563	433	330	223	197	150	127	101	85	67	43	37	27											
866		770	527	404	316	250	176	156	131	107	87	70	46	30	23	25									
867			634	488	383	307	249	202	167	129	111	93	77	63	51	40	30	23							
877				567	452	364	295	243	201	168	140	105	91	81	68	56	45	37	29	21					
878					505	430	352	291	243	208	171	145	123	89	81	75	60	52	43	34					
888						454	406	339	285	240	204	174	148	127	109	93	80	62	57	45					
889							411	375	331	280	238	205	176	152	131	113	99	85	73	63					
899									345	317	275	237	204	177	154	134	117	102	88	77					
1033	250	174	117	79	52	32	16																		
1034	355	263	188	135	97	70	48	31																	
1044	500	356	267	190	142	107	79	56	41	28															
1045	670	476	330	262	163	141	120	93	72	53	40	29													
1055		596	440	321	257	203	165	137	94	80	62	47	35	25											
1056		738	550	418	328	249	200	169	137	111	89	64	57	45	34	25									
1066			660	510	400	319	250	199	173	143	115	97	79	65	46	30	22	25							
1067				616	485	390	319	252	217	172	146	126	106	87	74	62	49	40	31	20					
1077				716	56.	460	376	311	259	216	183	156	132	112	94	81	67	55	46	37	29	22			
1078					665	538	442	368	308	260	220	186	161	137	118	95	87	74	63	53	43	35	28	22	
1088							511	426	357	303	256	223	191	167	147	130	107	92	79	66	56	49	41	34	27
1089								492	414	354	303	261	225	193	170	152	133	114	99	87	75	65	55	47	40
1099										402	346	301	260	226	198	177	153	134	118	104	92	80	70	60	53
	6	7	8	9	10	11	12	13	14	15	16	17	18	19-	20	21	22	23	24	25	26	27	28	29	30

Loads Above Solid Lines - Shear—60 lbs. per Sq. In. or less
Loads Between Solid Lines - Shear—60-90 lbs. per Sq. In.
Loads Below Solid Lines - Shear—90-180 lbs. per Sq. In.



A canvass of architects, engineers and contractors in Southeast United States concerning their opinions of the F and A system after having made use of it revealed that particularly on small jobs, it saved construction time and cost less; no ceiling finish is required, and where not applied acoustics are improved; and a minimum of equipment is required.

One architectural firm in Winston-Salem,. North Carolina, said, "This office has used most all types of floor systems and we consider the F and A system for buildings of more than one floor to be one of the cheapest precast concrete floor systems in this section."

Another southern architect said, "We have found the F and A system is economical for use in one- and two-story buildings where the spans are between 15 and 25 ft and where size of unit panels (length, width, and depth) are repeated. A crew can install approximately 5000 sq ft in a day." In discussing a change in design from steel bar joist construction to the F and A system, this architect said, "experience has shown that steel bar joists installed over crawl spaces are apt to rust out and lose their strength in 10 or 15 years."

Quoting another reply to a questionnaire, "we attribute the success of the F and A system to the following: it is lightweight and requires no special equipment for erection. It is economical, costing no more than older and competitive systems, and in many cases from 10 to 20 percent less. It requires a minimum of shoring which is 100 percent reclaimable. It has excellent acoustical and some insulating properties. It goes up quickly. It may be left exposed for a finished ceiling, spray painted or plastered. We believe that another reason that most contractors like it, and use it repeatedly, is that it is practically a dry system."

Table 2 gives safe floor loads recommended by F and A.

JOISTILE

As the name implies, this floor system is made up of joists and tile. As indicated by Fig. 4, both joists and tile are precast. The ends of the joists are properly spaced on concrete block walls or on steel I-beam supports. The tile or lightweight aggregate hollow blocks of special design are then set between them resting on the flange of the inverted T-beam joist. The bottom face of the blocks are flush with the bottom of the joists, giving a flat ceiling. The top of the joist is at the same level as that of the block. The floor can then be used as a deck over which building operations proceed. Rubber tired wheelbarrows may be used to bring on the topping which bonds to the top surface of the tile and to the stem of the inverted T-beam or joist, forming a modified I-beam, which acts monolithically with the tile and topping and can be so treated in computing floor loads.

The filler tiles are four-core units 5⅝ x 7⅝ x 24⅞ in., and can be produced in one operation on the L-5 Lith-I-Block machine, or any block machine using 26-in. plain pallets.

The system is designed to carry 50 psf live load on clear spans up to and including 16 ft. Load tests conducted by Concrete Joist and Products, Inc.,

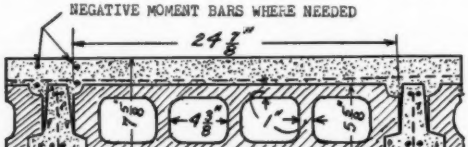


Fig. 4—JoisTile floor consisting of precast joist and filler blocks with cast-in-place concrete topping

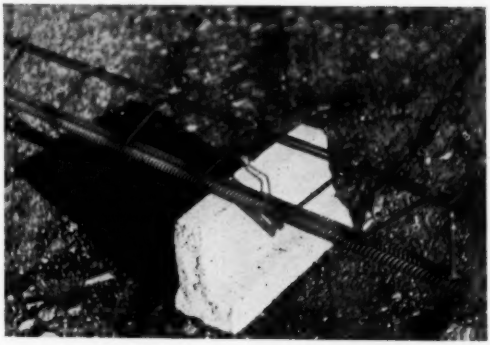


Fig. 5—JoisTile reinforcing truss

show that the floor system will pass the requirements of the American Concrete Institute for load testing of floors. The weight of the JoisTile floor system is approximately 53 psf including the 2½-in. slab.

Fig. 5 shows the welded reinforcing truss with diagonal stirrups. All reinforcing trusses used in the joists are welded in the plant itself. The size of reinforcing used, of course, varies with the length of joist and the load it is designed to carry. In JoisTile the size of reinforcing depends entirely upon the length, since the system is designed to carry a 50 psf live load only. Special joists are made with heavy reinforcing on shorter spans to carry heavier loadings if desired, but this is only on special order and is engineered in each case by the manufacturer.

Concrete used in the manufacture of the joists has a compressive strength of 4000 psi. Filler blocks or tiles used between the joists are made of lightweight aggregate, and the manufacturer's specification for this block is that a single unit shall be capable of carrying a load of 500 lb at the center when simply supported between joists.

The 2½-in. slab of 3000 psi concrete cast over the joists and filler tiles provides the equivalent of monolithic construction. Where the surface of the slab is to be the final wearing surface, it is finished with a steel trowel, but where the slab is to be covered it may be floated smooth to receive properly the type of floor covering specified. Resistance to negative moment may be provided by placing steel in the concrete topping either on top of the tile or in the chamfered space on either side of the top of the joist.

Recent tests show that the JoisTile floor can be used for warm air heating by passing the warm air through the cores of the filler block.

The simple design of JoisTile has many advantages, and offers an economical, fire safe and relatively permanent concrete floor.

Table 3 issued by Concrete Joists and Products, Inc., Chicago Heights, Ill., gives safe superimposed loads for spans up to 18 ft. Tests by recognized testing laboratories indicate that in slabs loaded in accord with ACI standards, deflection and recovery meet requirements.

TABLE 3—SAFE SUPERIMPOSED LOADS FOR STANDARD JOISTILE CONSTRUCTION*

DETAILS OF REINFORCEMENT

Clear Span, ft	Reinforcing A, No. and diam., in.		Safe superimposed load, psf	Spacing of web reinforcing, in.											
	Top	Bottom		A	B	C	D	E	F	G	H	I	J	K	L
8	2-¼	2-⅜	56	4	6	6	6	12							
	2-¼	2-½	162	4	6	6	6	12							
9	2-¼	1-⅜ 1-½	69	4	6	6	6	6	12						
	2-¼	2-½	111	4	6	6	6	6	12						
10	2-¼	1-⅜ 1-½	40	4	6	6	6	6	18						
	2-¼	2-½	75	4	6	6	6	6	18						
11	2-¼	2-½	48	4	6	6	6	6	6	18					
	2-¼	1-½ 1-⅝	82	4	6	6	6	6	6	18					
12	2-¼	1-½ 1-⅝	56	4	6	6	6	6	6	24					
	2-¼	2-⅝	82	4	6	6	6	6	6	24					
13	2-¼	1-½ 1-⅝	67	4	6	6	6	6	6	6	24				
	2-¼	2-⅝	90	4	6	6	6	6	6	6	24				
14	2-¼	2-⅝	70	4	6	6	6	6	6	6	12	18			
	2-¼	1-⅝ 1-¾	95	4	6	6	6	6	6	6	6	12	18		
15	2-¼	2-⅝	56	4	6	6	6	6	6	6	6	12	18		
	2-¼	1-⅝ 1-¾	77	4	6	6	6	6	6	6	6	6	12	18	
16	2-¼	1-⅝ 1-¾	62	4	6	6	6	6	6	6	6	12	18		
	2-¼	2-¾	79	4	6	6	6	6	6	6	6	6	12	18	
17	2-¼	2-¾	64	4	6	6	6	6	6	6	6	6	18	24	
	2-¼	1-¾ 1-⅞	85	4	6	6	6	6	6	6	6	6	6	12	18
18	2-¼	2-¾	53	4	6	6	6	6	6	6	6	6	18	24	
	2-¼	2-⅞	86	4	6	6	6	6	6	6	6	6	12	12	18

*8-in. modular depth (5⅝ filler plus 2¾ in. concrete).
Superimposed load is the total allowable load, not including the weight of the slab.
Dead load equals 50 psf including 2¾-in. slab.
Add dead load to safe superimposed loads given in table, and by proportion figure safe total loads allowable for other steel areas.
All joists are figured 7 in. longer than the clear span distance except for special conditions.
Web reinforcement consists of ¼ x 6¼ in. welded to top and bottom reinforcing to form trusses with diagonal web reinforcement.
Use double joists with or without special reinforcing or increase depth of slab when required under partitions or around openings in the slab.
Spans over 12 ft require a temporary continuous support under the center of the span until the 2¾-in. slab has set up. Provide camber with the support.
Thoroughly dampen joists and filler tile before placing slab concrete.

JoisTile floor systems are being installed in the Chicago area at a cost of less than $1.00 psf which is said to be competitive with any other system used in the area.

On an FHA project of six apartment houses and buildings in Hammond, Ind., JoisTile costs were approximately as follows:

JoisTile and filler blocks delivered	$0.40 per sq ft
Installation	0.07 per sq ft
Grout, topping and finish	0.31 per sq ft
Total	$0.78

STRESTCRETE

The term Strestcrete covers floor, roof, and wall slabs. The slab as made and used in floors and roofs will be briefly described. Fig. 6 shows the lightweight block used in the assembly of the beam or slab. Note the grooves on the side near the top and bottom for the reinforcement.

The blocks are steam cured for 8 to 12 hours and air cured for 28 days. They are then ground on the faces normal to the hollow core to produce uniform parallel bearing surfaces between the block.

A steel plate at each end of the slab, drilled to take the threaded reinforcement, brings the blocks together in compressed relationship at the steel level when the end nuts are tightened with a torque wrench. Ordinarily the rods are stressed to about 18,000 psi, although Strestcrete panels have been made and tested with the steel stressed to 54,000 psi, and it is understood that further studies are planned.

At this stage the slab, usually 2 blocks, or 32 in., wide, is lifted by a sling fastened to its ends and placed on a truck for delivery to the job. It is lifted by a crane from the truck by means of a similar sling and placed in position on bearing walls or girders. One thousand sq ft per hour of floor can be laid, ready for grouting between blocks over the moment steel or for grout

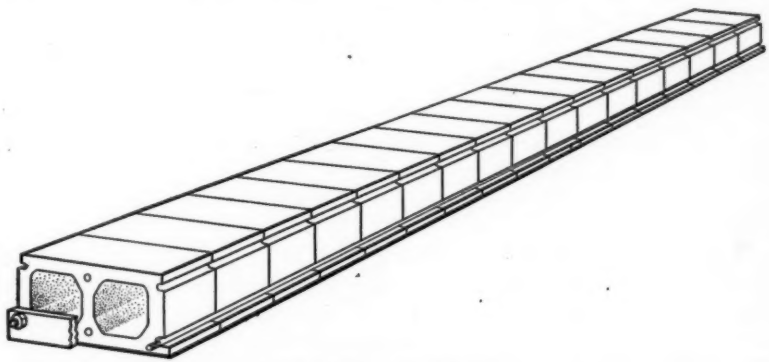


Fig. 6—Strestcrete slab unit

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TABLE 4—DESIGN DATA FOR LOADS AND SPANS—STRESTCRETE SLABS WITHOUT CONCRETE TOPPING

3" STRESTCRETE SLAB WITHOUT CONCRETE TOPPING

Dead Weight of Slab 20 lbs. per sq. ft.

Bottom Prestressed Steel	2-5 16" Ø	2-3 8" Ø	
Resisting Moment per ft. of width	4200" #	5925" #	
Top Negative Steel	1-3 8" Ø	1-1 2" Ø	
Resisting Moment per ft. of width	2375" #	6150" #	
Allowable Shear per ft. of width	356 #	356 #	
Moment Condition	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$
Type of Span	Simple	Continuous	Simple	Continuous
Span 5 ft.	92	115 115	122	122 122
" 6 ft.	57	73 73	89	96 96
" 7 ft.	37	48 48	60	80 81
" 8 ft.	23	32 32	41	57 69

4" STRESTCRETE SLAB WITHOUT CONCRETE TOPPING

Dead Weight of Slab 26 lbs. per sq. ft.

Bottom Prestressed Steel	2-5 16" Ø	2-3 8" Ø	2-1 2" Ø
Resisting Moment per ft. of width	5625" #	8100" #	13,050" #
Top Negative Steel	1-3 8" Ø	1-1 2" Ø	1-5 8" Ø
Resisting Moment per ft. of width	4725" #	8325" #	12,900" #
Allowable Shear per ft. of width	507 #	507 #	507 #
Moment Condition	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$
Type of Span	Simple	Continuous	Simple	Continuous	Simple	Continuous
Span 6 ft.	78	104 105	124	143 143	143	143 143
" 7 ft.	50	69 70	84	111 119	119	119 119
" 8 ft.	33	47 47	58	79 100	101	101 101
" 9 ft.	28	33 32	40	57 74	81	86 86
" 10 ft.			28	41 55	61	75 75

5" STRESTCRETE SLAB WITHOUT CONCRETE TOPPING

Dead Weight of Slab 42 lbs. per sq. ft.

Bottom Prestressed Steel	2-3 8" Ø	2-1 2" Ø	2-5 8" Ø
Resisting Moment per ft. of width	13,200" #	23,250" #	36,150" #
Top Negative Steel	1-1 2" Ø	1-5 8" Ø	1-3 4" Ø
Resisting Moment per ft. of width	12,725" #	21,450" #	30,900" #
Allowable Shear per ft. of width	1060 #	1060 #	1060 #
Moment Condition	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$
Type of Span	Simple	Continuous	Simple	Continuous	Simple	Continuous
Span 7 ft.	137	182 227	261	261 261	261	261 261
" 8 ft.	95	129 164	200	223 223	223	223 223
" 9 ft.	66	93 121	149	193 193	193	193 193
" 10 ft.	46	68 90	113	151 170	170	170 170
" 11 ft.	30	48 67	86	119 135	151	151 151
" 12 ft.		34 49	65	92 106	125	135 135
" 13 ft.		23 36	49	72 84	100	121 121
" 14 ft.			37	56 67	80	109 109
" 15 ft.			26	44 53	65	91 95

6" STRESTCRETE SLAB WITHOUT CONCRETE TOPPING

Dead Weight of Slab 48 lbs. per sq. ft.

Bottom Prestressed Steel	2-3 8" Ø	2-1 2" Ø	2-5 8" Ø
Resisting Moment per ft. of width	13,650" #	24,000" #	54,000" #
Top Negative Steel	1-1 2" Ø	1-5 8" Ø	1-3 4" Ø
Resisting Moment per ft. of width	19,650" #	30,375" #	43,875" #
Allowable Shear per ft. of width	1462 #	1462 #	1462 #
Moment Condition	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$
Type of Span	Simple	Continuous	Simple	Continuous	Simple	Continuous
Span 8 ft.	156	207 259	314	318 318	318	318 318
" 10 ft.	83	115 148	184	242 244	244	244 244
" 12 ft.	48	66 80	113	153 163	196	196 196
" 13 ft.	29	48 60	89	123 131	165	177 177
" 14 ft.		35 52	70	99 107	135	161 161
" 15 ft.		24 30	55	80 87	112	147 147
" 16 ft.			42	65 70	92	123 123
" 17 ft.			32	52 57	76	103 103
" 18 ft.			23	41 45	63	87 87
" 19 ft.				32 36	51	73 73
" 20 ft.				24 27	42	61 61

8" STRESTCRETE SLAB WITHOUT CONCRETE TOPPING

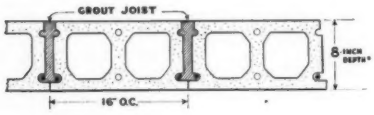
Dead Weight of Slab 56 lbs. per sq. ft.

Bottom Prestressed Steel	2-1 2" Ø	2-5 8" Ø	2-3 4" Ø
Resisting Moment per ft. of width	45,000" #	69,000" #	89,500" #
Top Negative Steel	1-5 8" Ø	1-3 4" Ø	1-1 8" Ø
Resisting Moment per ft. of width	39,600" #	57,000" #	77,250" #
Allowable Shear per ft. of width	1806 #	1806 #	1806 #
Moment Condition	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$
Type of Span	Simple	Continuous	Simple	Continuous	Simple	Continuous
Span 10 ft.	244	305 305	305	305 305	305	305 305
" 12 ft.	152	204 219	246	246 244	244	244 244
" 14 ft.	97	135 146	178	202 202	202	202 202
" 16 ft.	61	90 98	123	166 166	170	170 170
" 18 ft.	36	59 66	86	119 119	126	144 144
" 20 ft.	19	37 43	59	86 86	91	124 124
" 21 ft.		29 33	48	73 73	77	111 116
" 22 ft.		21 25	39	61 61	65	96 103
" 23 ft.			30	51 51	55	83 90
" 24 ft.			24	43 43	46	72 78
" 25 ft.				35 35	38	62 67

12" STRESTCRETE SLAB WITHOUT CONCRETE TOPPING

Dead Weight of Slab 62 lbs. per sq. ft.

Bottom Prestressed Steel	2-1 2" Ø	2-5 8" Ø	2-3 4" Ø
Resisting Moment per ft. of width	55,800" #	87,300" #	115,500" #
Top Negative Steel	1-5 8" Ø	1-3 4" Ø	1-1 8" Ø
Resisting Moment per ft. of width	48,750" #	70,125" #	93,750" #
Allowable Shear per ft. of width	2223 #	2223 #	2223 #
Moment Condition	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$	$\frac{wl^2}{8}$	$\frac{wl^2}{10}$ $\frac{wl^2}{12}$
Type of Span	Simple	Continuous	Simple	Continuous	Simple	Continuous
Span 12 ft.	196	261 276	308	308 308	308	308 308
" 14 ft.	127	175 186	234	255 255	255	255 255
" 16 ft.	83	119 128	165	216 216	216	216 216
" 18 ft.	52	81 88	117	154 154	175	184 184
" 20 ft.	31	54 59	83	113 113	130	160 160
" 22 ft.		34 38	58	82 82	92	131 131
" 24 ft.			39	59 59	68	100 100
" 26 ft.			24	41 41	51	76 76
" 28 ft.				27 27	36	57 57
" 30 ft.					23	42 42



VALUES GIVEN IN TABLES ARE SAFE SUPERIMPOSED LOADS IN POUNDS PER SQUARE FOOT.

NOTE: Values above and to the right of heavy line, produce a shear stress of 60 pounds per sq. inch.

*STRESTCRETE is manufactured in 3, 4, 6, 8, 10, and 12-inch depths and 16-inch widths.

$f_s = 20{,}000$
$f'c = 2{,}000$
$fc = 900$
$vc = 60$
$n = 60{,}000$
$\frac{}{f'c} = 30$

and floor topping if desired. Shear steel may be placed in the space between the blocks where necessary.

The blocks are being made for roof and floor slabs in 3, 4, 6, 8, 10, 12, and 16-in. depths.

Bending moment tests on 25 slabs 32 in. wide, on spans varying from 8 to 20 ft, and shear tests on slabs with 8 ft spans have been made by H. M. O'Neil Co. of Oakland, Calif. In his conclusions Mr. O'Neil says:

1. Panels behaved in a manner which justifies analysis of Strestcrete slabs by use of the ordinary assumptions commonly used in reinforced concrete design. Test results justify considering poured joists and topping (grout between the blocks, around the

steel, and over the top) as acting monolithically with Strestcrete units for purpose of design.

2. Shear strength of panels was high and ordinarily would not be a governing factor.
3. Plastic flow deflections under sustained load were small and of about the same magnitude that would be expected for a similar monolithic construction.

The writer visited the testing laboratory of the Illinois Institute of Technology to witness shear tests being made on panels 32 in. wide and with a 6 ft span, reinforced at the bottom with four ⅝-in. round rods. They were made of 6-in. block covered with a 1½-in. topping. Failure occurred by a diagonal crack cutting across the vertical joint between the end block and the one adjacent to it under a total transverse shear force of 16,000 lb for the 32 in. width or 6000 lb per ft width of slab. The allowable shear per foot width as given in Strestcrete design tables for slabs made of 6-in. block with no topping is 1060 lb per ft of width. Assuming the topping of 1½ in. or 18 sq in. to be capable of taking 60 psi, the topping alone would take 1080 lb. There was no slipping between the blocks along the ground faces, nor longitudinal shear failure between topping and block although the topping had a diagonal shear failure due to the high compressive stress.

Strestcrete loadings for 3, 4, 6, 8, 10 and 12 in. deep block without topping for spans varying from 8 ft for 3-in. block to 30 ft for 12-in. block are shown in Table 4. Note the control design values at the bottom. Using n as 30 indicates an E_c of about 1,000,000.

The initial tension in the steel puts a small camber in the slab. Test reports show that the deflection under design loads falls within ACI 318-51 permissible limits.

Comparative costs are not easy to make, but Ralph Adams, who manufacturers Strestcrete slabs in Ypsilanti, Mich., and who has laid many thousand

TABLE 5—APPROXIMATE COSTS OF STRESTCRETE PANELS*

Depth of Strestcrete, in.	Cost of completed standard panels at plant			Delivered, erected and joints grouted			Ordinary reinforced concrete, Mean 1950, ††per cu yd
	Per sq ft	Per cu ft	Per cu yd	Per sq ft	Per cu ft	Per cu yd	
6†	$0.80	$1.66	$45.00	$1.02	$2.04	$55.00	$60
8‡	1.05	1.56	42.00	1.28	1.92	52.00	
10§	1.30	1.52	41.00	1.57	1.89	51.00	
12**	1.50	1.50	40.00	1.85	1.85	50.00	

*Data prepared by Ralph Adams, Sr. Includes material, haulage and direct labor as of October 1950. Costs are for floor panels given here and ringed on Table 4. For panels other than shown with more or less reinforcement costs will vary slightly.
†6-in. panels 12 ft 6 in. long. Design superimposed load 92 psf.
‡8-in. panels 15 ft 7½ in. long. Design superimposed load 112 psf.
§10-in. panels 19 ft 4½ in. long. Design superimposed load 108 psf.
**12-in. panels 24 ft 4½ in. long. Design superimposed load 75 psf.
††Mean for 1949 was $55 per cu yd. General conditions since July 1 will raise mean to probably $65.

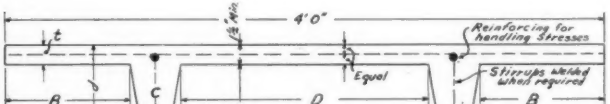
square feet of Strestcrete flooring, drew up Table 5 which gives a general idea of the savings possible. It appears that in the concrete alone there is a saving, but most architects, engineers, and contractors mention time saving, lack of delay due to cold weather, freedom from excess equipment and personnel as the most important money saving items. They point out that with the steel or concrete framework up, floors can be laid at any time, at any level, and can be used as a working deck just as fast as the panels or slabs are in place.

Clyde Stewart, manager of the Illinois Brick Co. concrete block plant says "We have no accurate cost analysis of competitive systems such as flat slab, pan construction, or other prefabricated systems, but there are indications that our present quotations are perhaps 5 to 10 percent lower."

FARHAN SLAB

Fig. 7 shows a cross section of the Farhan slab with notes as to ACI controls and design factors. It has the advantage, along with other reinforced precast floor units, of falling entirely within the limits of conventional reinforced concrete stress analysis and in being made under rigid factory control; giving a product that should be uniform in quality and dimension-wise.

L. G. Farrant, reports that the Dade County, Florida, School Board has accepted the slab for three new county schools.



Dimensions, in.

A	B	C	D
6	10	4	20
8	9⅜	4⅜	19⅛
10	9⅛	5⅛	18⅝
12	9	6	18
14	8⅝	6⅝	17⅝

Fig 7—Cross section of Farhan unit. Notes: (1) Concrete to have a compressive strength of 3750 psi at 28 days (ACI 711-46, Sec. 23), (2) Concrete cover ⅝ in. or equivalent to bar diameter whichever is larger (ACI 711-46, Sec. 11), (3) Reinforcement intermediate grades complying with ASTM 15-39, A16-39 or A82-34 (ACI 711-46, Sec. 22) and deformed in accordance with ASTM A305, (4) Where test loads are required they shall be 2.25 times design LL (ACI 711-46, Sec 40a)

Design Factors

f_c' = 3750 psi (ACI 318-51, Sec. 305a)
f_c = 1688 psi (ACI 318-51, Sec. 305a)
u = 350 psi (tensile reinforcement) or 245 psi (compressive reinforcement) (ACI 318-51, Sec. 305a)
v = 113 psi, or 450 psi (with properly designed web reinforcement (ACI 318-51, Sec. 305a)
n = 8
B (flange thickness) = 6 = 9 in. max. (ACI 318-51, Sec. 705b)
D (flange thickness) = 16 = 24 in. max. (ACI 318-51, Sec. 705a)
Span length without lateral support = 2 x 32 = 64 ft (ACI 318-51, Sec. 704a)
f_s = 20,000 psi (ACI 318-51, Sec. 306)
d = effective depth
t = flange thickness
b = stem width
A_s = area of tensile reinforcement

Discussion of a paper by F. N. Menefee:

Advances in Precast Floor Systems*

By RALPH W. ADAMS, SR., F. A. BIBERSTEIN, ALBYN MACKINTOSH, O. NEIL OLSON, HUGH M. O'NEIL and AUTHOR

By RALPH W. ADAMS, SR.†

In the introduction of the excellent and informative article by Professor Menefee, floor systems (b) and (c) are adaptations of "old" floor construction methods while systems (a) and (d) are relatively recent in conception. Type (d) differs from the others in being fabricated entirely of prestressed materials. All of the described systems have the effect of reducing the cost of falsework or eliminating it entirely. The present interest of architects, engineers and the public in the newer types of construction is comparable with that of half a century ago when reinforced concrete in various forms first came into general use.

Engineers of that period had to reduce to usable form the data developed by theory and realized in practice. It is still the obvious responsibility and qualification of the structural engineer that he be able to develop guiding principles of design from observed physical characteristics of materials and industrial processes made available for his use. In this approach it is well not to depend on arbitrary formulas which at best can be only approximations as to the effects of natural forces.

Strestcrete is a floor system of prestressed concrete, shop prepared in convenient panel form, for erection on the job. There has been an entirely needless air of mystery surrounding the whole subject of prestressed concrete and the writer believes that with known physical data any competent structural engineer can design with assurance in Strestcrete. There are in fact more characteristics within close control of the designer in this type of construction than with reinforced concrete in any other form. The basic concrete units are molded to uniform dimensions, cured under laboratory conditions and face ground with machine precision. Likewise all reinforcing is positioned in design location with accuracy and with no greater tolerances than those customary in structural steel fabrication. Prestress of both steel and concrete is fully within control of the designer and is applied by instruments.

*ACI Journal, Oct. 1951, p. 113. Disc. 48-8 is a part of copyrighted Journal of the American Concrete Institute, V. 24, No. 4, Dec. 1952, Part 2, *Proceedings* V. 48.

†Consulting Engineer, Ypsilanti, Mich.

Strestcrete is peculiarly adaptable to research study and it is possible to prepare a panel, subject it to any range of prestress within the strength of the materials and observe effects of load under varying conditions. It may then be grouted and loadings continued.

Information secured by many tests which would appear to be of use to the engineer in arriving at a design is as follows.

First—A panel of Strestcrete units may be laid side by side, reinforcing added (without prestress), and the usual joint grout and topping applied. Such a panel has no strength until the grout has hardened and cannot be built without forms, but when cured it will carry such loads as may be anticipated when evaluated by current theory as a reinforced concrete slab. Resistance to both shear and moment appear unaffected by the vertical joints between adjoining block.

Second—Just before grouting, the dry assembly of block and steel elements may be subjected to an end thrust by drawing up the bolts against end washers. The thrust is imposed eccentrically so that induced moments are opposed to those caused by gravity loads. In theory an exterior force could be used instead of the bolts. As the pressure is applied a point is reached where the panel is so tightly squeezed that it supports itself over the entire length. It can be lifted by tackle, transported as desired and placed upon prepared permanent supports. Grouting may be applied at any convenient time thereafter permitting erection in subfreezing temperatures (Fig. A).

Third—In the dry assembly, the panels are composed of prisms of concrete acting as struts under the end thrust imposed by the prestress. It will however support design or double design loads while the elements can act only as struts. Under such loads the whole cross section of the block must resist all shear stresses as there is nothing else to resist such forces. It is then apparent that the grout joint later provided cannot be considered as the principal carrying member with block acting only as fillers. The whole block is structurally active in resisting moments and shears caused by applied loads.

Final operations after erection are grouting of joints and the application of topping where required. Both add to panel strength as might be ex-

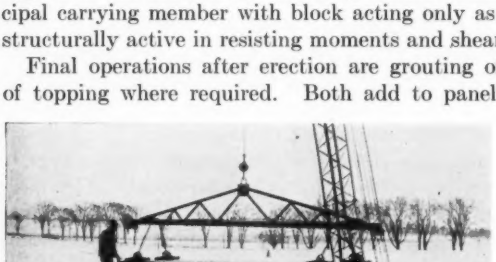


Fig. A—Setting 10-in. Strestcrete panels, 23 ft 1½ in. long, in freezing weather

pected from added amounts of material and increase in depth of section. Final developed stresses in joint grout and topping are low since they are zero under dead load conditions.

Fourth—Subsequent loading of completed prestressed panels show that they will carry the same ultimates as similar panels which have no prestress. From this it may be deduced by the designer that Strestcrete should have as an arbitrary minimum the same main reinforcement as indicated by current theory applied to similar but unstressed panels.

CONCLUSIONS

There is a fairly definite field or range of prestress beginning with zero and having an upper limit not yet determined where the designing engineer may seek and obtain better characteristics of completed panels through prestressing. Of principal importance economically is that under an end pressure of about 3 to 5 tons per sq ft of gross section, the panels are stiff and so strong that they can be handled and installed without centering of any kind. Design or construction loads can be applied immediately. Minor but consistent improvements in other structural characteristics at design range of loads are also apparent. Thus deflections are less than would be expected with other common types of construction.

Probably the basic advantage to the general contractor is the fact that when he uses prepared panels of this description he no longer has to manufacture his floors on the site. The field operation becomes one of assembly where cranes rapidly and easily swing large sections into position in a matter of minutes. A messy, wasteful, and time consuming operation is thus reduced to a few hours of erection and other trades may proceed without interruption. The possibilities here have yet to be fully explored.

It may be of interest to the analyst as to whether the initial squeeze imposed upon the block or other type of prestressed concrete is indefinitely retained, increased, or diminished by effects of time, fatigue, or changes in temperature and moisture content. Undoubtedly all of these have minute effects of some kind. Professor Menefee seems inclined to believe "that most of this original prestress is lost through shrinkage and plastic flow, etc." This belief may stem from observations of other types of so-called prestressed concrete where highly strained steel is incorporated in wet concrete and left to the uncertain range of shrinkage incident to curing. Over a three-year period at the plant of Adams Concrete Products Co., Ypsilanti, Mich., all completed work has been under observation with special attention to effects traceable to loss of prestress. In the aggregate there is noticeable only a minute deflection of about 1/1000 of the span due to any effects of this nature. A slight camber built into the piece is retained. Routine manufacturing methods include an initial prestress on the assembly table with the panel then being stored in this condition. There is a setting to stress fixed by the designer just prior to installation.

It has already been shown that with or without prestress the panel will carry its design or ultimate loads. Complete retention of prestress is therefore of relatively minor importance. Where desired, a factor of anticipation may be made at the time of final setting of the bolts. If as a considered possibility the initial prestress diminishes in time to the vanishing point as supposed by Professor Menefee, it has already served its principal purpose and the panel is still an excellent type of reinforced concrete construction with all main steel members anchored firmly to end plates and having other desired structural characteristics.

It is evident that the basic units or block may be composed of any material which has suitable characteristics when ground. Natural aggregates can be improved upon and lightweight manufactured materials of specified qualities are being produced and will undoubtedly be used extensively.

By F. A. BIBERSTEIN*

The informative paper by Professor Menefee gives an enlightening survey of the current status of precast floor systems including the Dox system with which the writer has had some experience.

Professor Menefee points out that "for the most part the standard reinforced concrete theory is applicable to these new floor systems." In this connection, it might be of interest to discuss certain details associated with properties of the cross section of a Dox system unit having the geometric outline illustrated in Fig. 1.

NET CROSS SECTIONAL AREA

The gross cross sectional area of 6 in. deep Dox-type units used in flexural test planks at Nabco Plank Co., Washington, D. C., may be taken as 6×16 in. = 96 sq in. The net area, as determined by the immersion weighing method outlined in ASTM C 140-39, was found to be 56 sq in. or approximately 58 percent of the gross area.

An average net area of 59 sq in. was found for similar units by deducting from the gross cross sectional area the sum of (a) the average area of the three tapered core spaces and (b) the area of the two grooves. The grooves provided in the underside of the block to accommodate the steel rods and their bonding grout were not tapered.

On the assumption that the metal cores used in the manufacture of the block were tapered uniformly, their volume displacement was determined, by immersion weighing, for a block 7.87 in. long in the direction of the core-space axis. The average cross sectional area of the cores was obtained as the quotient of the core volume and the length, 7.87 in. The average net area of the block was taken to be the difference between the area contained by the geometric outline of the block and the average section area of the cores. The

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average net area, 59 sq in., obtained in this way is approximately 61 percent of the gross area, 96 sq in.

CENTROIDAL AXIS

The theoretical distance in a 6-in. plank from the center of ⅝-in. diameter steel rods, resting on short shelves projecting ¼ in. beyond the invert of the grooves, to the outer surface of a 1.5 in. thick concrete topping is 6.56 in. (To reduce the number of variables, discussion is limited to planks containing two ⅝-in. diameter rods and a 1.5 in. thick concrete topping.)

Position of the centroidal axis of the cross section was determined with the help of the "transformed" section method and on the assumption that the net transformed area of the unit between the centroidal axis and the underside of the topping was not significant.

Areas of component materials of the section—gravel-concrete topping, cinder-concrete block, steel rods and grout—were considered replaced by corresponding areas of concrete having an ultimate compressive strength of 3000 psi and a modulus of elasticity of 3,000,000 psi. Exceptionally, masonry material on the tension side of the centroidal axis, including the rod bonding grout, was excluded from the centroid calculation.

Let u = distance, centroidal axis to top of cinder block (underside of topping), in.
n = modular ratio, steel and gravel concrete, assumed as 10
m = modular ratio, cinder- and gravel-concrete
d = effective depth, 6.56 in.
t = topping thickness, 1.5 in.
w = untransformed width of cinder block, in.
A = area of two ⅝-in. diameter rods, 0.6 sq in.

Then, since the moments of "homogeneous" areas vanish about their centroidal axis, the general expression for a value of u, including the moment of the transformed cinder block area between the centroidal axis and the topping, becomes,

$$(16t)\ (u + t/2) + mu^2w/2 - An(d - t - u) = 0$$

from which

$$u = 0.412 - mu^2w/60$$

which suggests that the centroidal axis may be found within the upper 1-in. shell of the cinder block, under the topping, where the width w is constant.

The maximum value of u, 0.412 in., is altered little by the second term (which takes into account the compression in the cinder block) for, giving u in the second term a maximum value (0.412 in.) and with a maximum width w of 16 in., u is reduced by 10 percent if m is assumed to be unity. However, the modular ratio of cinder- and gravel-concrete probably has a value between ½ and ¼. Thus, u will always be less than 0.412 in. because the second term is negative but the reduction will probably be about 3 percent.

Solving the general expression for u, assuming m has the values ½, ⅓, and ¼, one finds, for $n = 10$, and for

$m = 1/2$, $u = 0.392$ in.
$m = 1/3$, $u = 0.398$ in.
$m = 1/4$, $u = 0.401$ in.

If the transformed area of the cinder block for a depth of approximately 0.4 in. is included, the compression area is increased (from 24 sq in.) by approximately 10 percent. This relatively large increase, however, is not significant. The moment arm of the internal couple jd varies by approximately 1 percent as an effect of the additional compression area. Furthermore, the planks are customarily under-reinforced and, practically, only the steel moment would be affected by variation in jd.

Neglecting the compression area in the transformed "stem" of the cinder block under the topping, the centroidal axis would be 1.91 in. below the outer surface of the topping, or $kd = 1.5 + 0.41 = 1.91$ in.

MOMENT OF INERTIA

Moment of inertia of the cross section of a reinforced concrete beam is defined as: (a) moment of inertia of the transformed beam cross section while tensile bending stresses are less than the ultimate tensile strength of the "homogeneous" masonry material, and (b) moment of inertia of the transformed beam cross section after tensile resistance is shifted to the steel rods by failure in the masonry part. The latter is probably more significant and what follows refers to that definition. In its determination, it is supposed that sufficient cracks have been developed which, extending to the centroidal axis, would justify the assumption that the tensile strain of deflection is supported solely by the rods.

The moment of inertia of the transformed section may be required in connection with a study of plank deflections. Experience has shown that load-deflection graphs for simply supported 6-in. Dox-system planks, composed of 6-in. Dox-type cinder block, two 5/8-in. reinforcing rods and a 1.5 in. thick gravel-concrete topping, have a substantially linear portion following a variable transition curve from zero origin to a value of load at which the failure of the masonry parts in tension is completed and additional tensile resistance is assumed entirely by the rods.

If a straight-line function is fitted to the experimental data, analytically or graphically, it is interesting to note that the empirical slope agrees well with the slope indicated by elementary flexure theory.

Letting k represent a constant whose value depends on the type of loading, maximum deflection for a simply supported plank under central or symmetrical loading may be expressed as

$$D = k P L^3 / E I$$

in which I is the moment of inertia of the transformed section and E the modulus of elasticity of the material into which the various component materials have been transformed. The slope of this function is constant and given by

$$P/D = E I / k L^3 \text{ lb per in.}$$

In a series of flexure tests made at Nabco Plank Co. during 1951, the greatest variation between the theoretical and experimental slopes was found to be approximately 12 percent. In some cases there was no apparent variation. It was also found, however, that comparisons of experimental and theoretical slopes could only be made if intermediate grade steel rods had been used for reinforcement. There was no well-defined linear trend in the load-deflection graphs obtained for planks which contained re-rolled rail steel reinforcing rods.

In general, the moment of inertia, for $n = 10$, may be given by the expression,

$$I = 16(1.5)^3/12 + 16 \times 1.5(0.75 + u)^2 + 6(5.06 - u)^2 + 16mu^2/3$$

If the "stem" transformed area is included, the last term is used and u would have a value corresponding to the value assumed or determined for m. If the "stem" is not included, the last term is excluded and u would have the value 0.41 in. with a corresponding value for $I = 166.5$ in.^{4}

Corresponding with values of m of $\frac{1}{2}$, $\frac{1}{3}$, and $\frac{1}{4}$, the last term in the general expression for moment of inertia has the numerical value 0.15, 0.10, and 0.08, respectively. Moment of inertia of the transformed areas of the steel rods about their "gravity" axis is approximately 0.20 in.^{4} and is not included. If contributions to the moment of inertia by the transformed "stem" and steel rod areas (about their centroidal axis) are not neglected, the moment of inertia is increased by approximately 0.10 percent. Greatest contribution to the moment of inertia comes from the area of the topping and the transformed area of the steel rods about the centroidal axis which is located in the upper 1-in. shell of the cinder block.

Prof. O. Neil Olson suggested to the writer the possibility of using a semigraphical method for calculation of moment of inertia, which, he pointed out, was described in "Stress Analysis and Design of Elementary Structures," by James H. Cissel, 2nd Edition, 1948, p. 333. This method, it is believed, would be quite useful if the location of the centroidal axis (and the corresponding value of I) is desired for the entire cross section, or if it is known beforehand that the centroidal axis will be somewhere below the upper 1-in. shell of the cinder block where the net width of the block is no longer constant. The net width of the Dox-type cinder block varies from top to bottom and calculation of the net width as a function of the depth is hardly worth while. Using the semigraphical method for the conditions stated, the moment of inertia was found to have a value of 165 in.^{4} It is believed that the arithmetical method will be found more convenient if variations in the net block width are not involved. If they are, the semi-graphical method is preferable. Consideration of the deflection of the plank in the early stages of loading—before tensile strength of the masonry is exceeded—would require the use of a value of I based on the entire cross section; for this case the semi-graphical method will be found less burdensome and sufficiently accurate for calculation of the full-section moment of inertia.

By ALBYN MACKINTOSH*

Professor Menefee's paper raises several interesting questions in regard to engineering principles. In Strestcrete we note that surfaces of precast units are ground to obtain good bearing and ground surfaces are perpendicular to the longitudinal axis of beam units. No mortar is placed between these surfaces. The manufacturer states that "ACI T-beam formulae are used. Unit stresses of concrete and steel are those normally used for concrete and steel of equal strength." It seems that the effect of prestressing is neglected in design, and stresses used are those of cast-in-place concrete and not concrete masonry.

Is it right to neglect prestressing in design when this prestress introduces tensile stresses in the top of the beam near the supports which are not relieved by vertical loading, and what happens to shear at such points? It is pointed out by Professor Menefee that this prestress is large enough to put a camber in the slab.

An analysis of the charts and shear values given show that the portions of the units adjacent to grout fills are included in design for shear, which means that the friction developed on ground surfaces through prestressing is carrying considerable shear. Charts are set up for a slab without concrete topping. Are we to assume that a grout fill of about 1½ in. in width is sufficient to provide bond for the steel and precast units and develop the shear and T-beam action in the floor system? Are we to start designing beams using friction as a force to resist shear?

The writer analyzed the tests supervised by H. M. O'Neil and feels that these tests do not justify the stresses claimed, and were not analyzed nor conducted in a way which clears up the weaknesses expected in this type of construction.

The writer will be interested in the story that long-time tests will reveal concerning this method of construction. At the present he agrees with the Los Angeles City and other authorities in this area who have refused to accept this system of construction until a rational analysis in accordance with well established principles of mechanics is determined, and long-time tests have established the results in loss of prestress, stress redistribution, progressive cracking and slipping between block.

By O. NEIL OLSON†

Professor Menefee's paper is most timely in that it draws attention to various precast floor and roof systems which have become popular since publication of ACI 711-46, and which, together with I-beam and hollow-core joists, offer a solution to some of the problems of building construction during periods of material shortages and high labor costs. It is interesting to note that two of the four best known systems are fabricated from concrete

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units made on an ordinary block machine, thus making them potentially available in all areas having concrete block plants.

The writer believes that the weight given for the 8-in. Dox plank should have been given for the 6-in. plank. The 6-in. plank weighs about 30 psf, and the 8-in. plank weighs about 50 psf, both untopped. Although it was not discussed in the paper, it should be noted that the Dox system has been produced with a 4-in. block, and is now being produced with both 6-in. and the 8-in. block. The 8-in. block offers the advantage of greater effective depth of section with corresponding higher loads or greater spans.

The load table presented for the 6-in. Dox plank should be considered as the load table from one producer only and should not be taken as the exact table for all producers of the system. Local and state regulations cause variations in the "approval loads" that the planks are rated for, although the basic design or test data may be almost the same for all. It is recommended that interested parties contact the nearest producer of this system, or of any of these systems, for the load table or chart which is applicable to that area. The writer has observed in three widely separated parts of the country that each group of building officials has placed a different interpretation on the design and test data with the result that varying approvals are granted.

By HUGH M. O'NEIL*

The tests on Strestcrete slabs, mentioned in the author's paper, were made by Basalt Rock Co., Strestcrete manufacturers and fabricators, Napa, Calif. With the help of Basalt's engineer, Don McCall, the writer's firm designed and supervised the tests. R. E. Copeland, director of engineering, National Concrete Masonry Assn., acted as consultant and assisted in setting up the test program.

It was the wish of Basalt Rock Co. to make an extensive series of tests, but so little was known about how the slabs would behave, especially in shear where neither cast-in-place joists nor topping were used, that it was necessary to run a few preliminary bending and shear tests to get data to be used in designing the final test program.

Some difficulty was encountered in making a test set-up that would produce shear failures in the preliminary tests. Unless points of load were quite close to supports, tensile steel failure would occur, even with the larger steel areas. No vertical shear failure occurred at ground-face joints in preliminary tests; all were by diagonal tension, even for dry test slabs having neither topping nor grout, which, due to lack of bond in the steel, were forced to act as flat arches. The preliminary tests indicated that flexural strength of the concrete units was high in comparison with strength of the steel in tension. Final tests substantiated this indication.

TEST DETAILS

The final test program consisted of testing to destruction 48 Strestcrete

*Structural Engineer, Oakland, Calif.

panels of various lengths and sizes, both in shear and bending. Two panels were made up as continuous beams with cantilever ends. Both bending and shear tests were made with and without topping and with and without cast-in-place joists. The term "grouted joist" was used to designate the latter type of joist. Topping, where used, was kept at a constant thickness of $1\frac{1}{2}$ in. for all tests.

All Strestcrete units for the test program were manufactured from identical concrete mixes and cured in an identical manner. Concrete for topping and grouted joists were of the same mix (5 sacks per cu yd) for all panels, except two shear test panels, for which a lower strength mix ($3\frac{1}{2}$ sacks per cu yd) was used. Test cylinders were made and tested for the standard and low strength concrete used for topping and grouted joists. Typical lightweight units were selected at random for each size unit used in the tests and these were subjected to a standard compression test. Small specimens were cut from several Strestcrete units and subjected to compression tests. Samples of each of the four sizes of reinforcing steel were selected and tested in tension.

Two of the bending test panels and two of the shear test panels had no prestress in bottom steel. Steel in panels without grouted joists was prestressed immediately before testing. Steel in panels with joists was prestressed at the time of placing of the joists; but immediately before testing, nuts were loosened with a wrench and screwed by hand "thumb tight" against end washers. All prestressing was done by tightening nuts on threaded ends of rods with a torque wrench to approximately 18,000 psi.

High strength end block made of ordinary (not lightweight) concrete were used at both ends of test panels to prevent crushing of concrete under end washers due to prestress in rods. End washers used were steel flat bars.

Loads were placed in various increments. Deflection readings were taken with an Ames dial for each increment of load until failure appeared imminent. On many of the tests, after each of the first several increments were added, the complete superimposed load was removed and deflection readings were taken to determine the amount of "permanent set." Two test panels were subjected to a repeated load applied 100 times. Three test panels were given a sustained load test to determine the effect of a long continued load on deflection. Although the amount of prestress in the tensile steel was relatively low (roughly 18,000 psi) it was found that it materially decreased the tendency for visible hair cracks to form under test loading.

TEST RESULTS

The following are general observations by the writer regarding test behavior.

(1) Use of ground faces had no noticeable effect on bending strengths and panels behaved much like monolithic concrete, even for test panels without topping or grouted joists.

(2) Topping and grouted joists acted integrally with ground face units and increased the strength of panels as would be expected for monolithic construction.

(3) Initial failure in all moment tests was due to tensile yield of steel. Concrete compression failures occurred only after panels had undergone excessive deflection due to steel elongation.

(4) Use of prestress in steel had no noticeable effect on ultimate bending strength or deflection at ultimate loads. Prestressing did reduce deflections considerably for lighter loads. The reduction of deflection due to prestressing was approximately 25 percent at superimposed loads equivalent to ordinary design loadings.

(5) Prestressing also had the effect of materially increasing the amount of load necessary to produce visible hair cracks.

(6) Shear resistance at the vertical ground face joints was high. No slipping at vertical joints occurred for panels having grouted joists, practically all shear failures being in diagonal tension, even for slabs having no topping and without prestress in steel. Slabs with neither grout nor topping failed either in diagonal tension or by vertical slip at joints. Prestressing bottom steel had no noticeable effect on shear strength for slabs having grouted joists.

CONCLUSIONS

The following are conclusions by this writer based upon results of the test program.

Panels behaved in a manner which justifies analysis of Strestcrete slabs by use of the ordinary assumptions commonly used in reinforced concrete design. Test results justify considering grouted joists and topping in acting monolithically with Strestcrete units for purposes of design.

Shear strength of the panels was high and would not ordinarily be a governing factor in design.

Plastic flow deflections under sustained load were small and of about the same magnitude that would be expected for similar monolithic construction.

The initial prestress used was not high and it probably would diminish to a much lower value over a period of time due to shrinkage and plastic flow; therefore the writer feels that it should not be considered in the structural design of the slabs. However, prestressing does benefit the slabs in several ways. Its primary purpose is to assist in assembly and erection of the slabs but it also undoubtedly aids in securing a denser and closer-fitting assembly during fabrication and in reducing shrinkage and plastic flow deflections after erection.

In a later test program under the writer's supervision, several Strestcrete panels were tested using higher strength steel (yield point approximately 90,000 psi, ultimate strength about 160,000 psi). In these tests SR-4 strain gages were used to determine tensile stress in the steel. It was found impractical to use a prestress in the $\frac{5}{8}$-in. round rods of much more than 40,000 psi because to do so caused crushing of the ends of the block in bearing under the steel plates. All failures in this test program resulted from initial failures in the steel, indicating that Strestcrete units are suitable for use with higher strength steel. As before, the slabs behaved in a manner similar to that which would be expected for monolithic construction.

SR-4 strain gages on the steel indicated that steel stresses during the test conformed closely to theoretical values. Strain gage readings also substantiated the correlation between the prestress in the rods and the torque wrench readings.

At the present time, the use of higher strength steel for Strestcrete construction seems impractical due to the higher cost, except in special cases where long spans with thin slabs are necessary.

Due to its low cost, dependability and speed of erection, Strestcrete has rapidly gained wide acceptance and use in the Oakland-San Francisco area.

AUTHOR'S CLOSURE

It is believed that most conservative engineers were skeptical of the "assembled concrete block" type of precast joist when first seen. The conventional beam had no vertical joint where sliding of one block up or down relative to its neighbor could take place, and without such a joint shear resistance prevented any vertical displacement. But probably no one ever saw a failure of the conventional beam in pure vertical shear. The shear stress was there, but it was accompanied by a longitudinal shear stress and the two combine to produce a diagonal tension by which failure often occurs, where beams are tested to destruction.

Probably those of us who were doubtful of the shear resistance of the "assembled concrete block" type forgot that the compression portion of the cross section would have, under its own dead load, a compression stress which when multiplied by the coefficient of friction, would provide substantial resistance to sliding; also that this amount when added to the shear resistance of the 2-in. grout between each row of block, in the case of Strestcrete, would provide satisfactory resistance in shear through resistance to diagonal tension. Then when greater loads come on the floor, the compressive stress becomes greater, the coefficient of friction probably also becomes greater and sliding along the joint does not occur. As pointed out by Mr. O'Neil most Strescrete floors or roofs have a top coating. This of course increases the shear resistance.

With specific reference to comments by Mr. Mackintosh, the author is of the opinion that prestressing of the amount used in the Strestcrete system (about 18,000 psi) may be neglected in design. There is, of course, some tension set up at the top of the joist throughout its length. Dead load alone will usually cancel this, except as Mr. Mackintosh points out, at the supports.

Mr. O'Neil suggests that the 18,000 psi prestress diminishes with time due to shrinkage and plastic flow; this seems plausible. If the prestress does disappear, tension at the top near the supports, not cancelled by dead load, will disappear too. If it doesn't, there may be a minute crack at the top of the joist between the three end block, let us say. The grout or top coat will fill these cracks and no harm results.

We must keep in mind that in ordinary concrete beam construction, we have grown used to the idea of knowing and ignoring that most of our longitudinal reinforcement is in compression to start with and the concrete at the steel level is in tension, where all the time our theory assumes no stress. When the dead and live load moment begins to function, the above tension in the

concrete is increased and often the strain is great enough to induce hair cracks over a considerable portion of the underside of a beam or joist.

Contrarywise, if at the time of placement in a structure, or shortly thereafter, shrinkage and plastic flow of the mildly prestressed concrete block joist reduces the prestress in the steel to almost zero, the stress situation is more ideal and more nearly in accord with what we assume in our ordinary stress formulas than if the steel is actually in compression and the concrete is in tension when the moment stresses due to loading begin.

As to shear near the supports, the author witnessed some tests in Chicago in which the final "diagonal tension" crack cut right across the vertical ground joint between two block. There was no slipping of one block on its neighbor. Nor was there any horizontal shear between the topping and the block. From that time on, the author has not worried about transverse shear movement between block near the end. If the friction develops enough resistance to sliding to cause failure by a diagonal tension crack in the block, the friction in psi was equal to the tensile strength of the concrete and that is all we ask of shear stress in concrete. From that point onward we put the burden on diagonal steel stirrups, if more shear resistance is needed.

The author did not make any of the charts or tables in connection with the paper, but he has checked many of these tables at random and is satisfied that for general guidance of engineers and architects they are all right. From experiments made years ago in connection with topping on the I-beam type of joist observing for T-beam action and from observing tests on Strestcrete, he is confident that grout longitudinally placed between the Strestcrete block will bond to the sides of the block and, with the topping, will form a T-beam with a stem equal to the width of the grout and all the ribs in the block. Also, if steel sufficient to carry the moment is prestressed to 18,000 psi, in most cases friction between the end block and its neighbor will crack the block diagonally before sliding.

The Dox system does not have the 2-in. space between rows of assembled block. They are keyed or interlocked which keeps them correctly lined up along the vertical longitudinal joint between the plank. But the plank when laid is a flat arch, the ends of which are anchored by tension steel and the whole cross section of the plank is in compression for light loads. That compression (normal to the right section) multiplied by the coefficient of friction, with some assistance from the grout around the steel and from the steel itself prevents sliding of one block relative to another along the right sectional transverse planes. As the load gets heavier the compression probably becomes less uniform but always greater in total, with a probable increase in the coefficient of friction.

While the foregoing analysis is qualitative, it could be made quantitative; and when all factors are considered we get the answers to why these assembled block types of precast joist fail by diagonal tension induced by shear and frictional resistance. As long as tests to failure show diagonal tension cracks

crossing the vertical joint, it does not matter whether the beam or plank or joist is made in the conventional manner or of assembled precast block.

Professor Biberstein has made tests on a series of twenty-one 6-in. deep planks made of cinder aggregate Dox block, with a gravel concrete topping of 1½ in. In a few cases he got a failure by horizontal shear, none by vertical sliding; for the most part they were by diagonal tension or by yielding of the reinforcement.

There is much more that could be said now of these advances in precast floor systems, and new ideas are springing up every day. A thorough study of shrinkage and plastic flow of individual concrete block made of various lightweight aggregates and with various methods of curing, and of the block assembled into plank is now in order.

In closing, the author wishes to thank the many persons who contributed to the collection of the information in the original paper and to the discussion following.

Manufacture and Use of Machine-Made Precast Structural Elements*

By A. G. STREBLOW†

SYNOPSIS

Experience with precast piles, joists and cored roof and floor sections and the introduction of automatic block making machines suggested the development of new precast products. The characteristics and application of floor and roof slabs, wall sections, prefabricated packaged buildings, columns, beams, planks and special structures are described.

INTRODUCTION

Experience in the field of conventional precast structural concrete has created a desire on the part of many concrete products manufacturers to improve production and reduce cost by utilizing the tremendous capacity of modern concrete block machines. This necessitates the assembly of relatively small precast elements into structural panels of reinforced concrete. These are used for floors, roofs, walls, columns, beams and arches.

Patents, which pre-date the design of machines for the manufacture of concrete blocks, have been issued in Germany, France, England and the United States for systems which originally employed clay tile as the structural elements. Floor systems assembled from tile with steel tie rods forming the tension members were in use in the United States as early as 1889. In recent years this type of floor has gained increased popularity. Numerous variations and improvements in design, materials and methods of assembly have evolved.

In all systems the application of load subjects the individual segments to a compressive stress. This is true whether the reinforcing steel is pre-tensioned or not. If the contacting surfaces do not have complete and uniform bearing, points of concentrated stress occur which invariably cause a fracture in the precast segment. Perfect bearing is therefore a necessary requirement for all slab systems assembled from a plurality of structural elements. This can only be provided by applying mortar between the units or by grinding the contacting surfaces.

Uniform application of mortar between vertical surfaces is not easily accomplished as a factory operation. For this reason, most manufacturers

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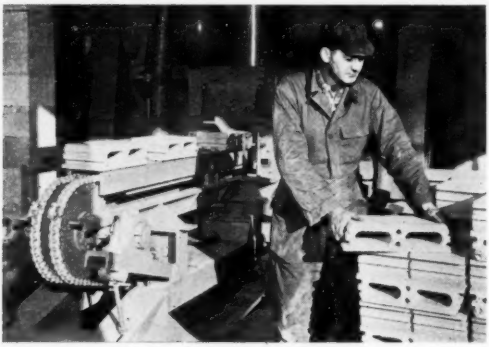

Fig. 1—Grinding 4-in. roof and floor slab units on a 30-in. double spindle grinder

of floor and roof systems assembled from individual elements have found it most economical to obtain good bearing between units by grinding.

All concrete can be ground but the operation is not commercially practical for extremely hard natural aggregates. Best results have been obtained with concrete composed of expanded clays and shales, cinders, pumice, limestone and expanded slag. Other materials usually contain such a large percentage of silica that the life of the abrasive is extremely short and consequently the cost of the operation is high.

For grinding, double spindle disc grinders are employed. These machines are now made in standard sizes for discs measuring 26, 30 and 42 in. in diameter. Each spindle is powered by a 25 to 60 horsepower electric motor, depending on the size of the disc, the maximum size of the unit to be ground, the amount of stock to be removed and the hardness of the concrete.

PRECAST UNITS

At the Napa, California, plant of Basalt Rock Co., Inc., three of these machines are now in operation grinding concrete units used for the assembly of Strestcrete floor and roof slabs, tongue and groove siding, integral steel frame buildings, studwall and silos. All concrete segments used by this company are manufactured on Besser Super Vibrapac machines. The aggregates used are volcanic pumice and Rocklite, an expanded shale. The precast units are cured by conventional low pressure steam methods, then air-cured and dried for a minimum of 28 days. As needed, the units are taken from stock and precision ground to accuracies of .002 in. for parallelisms and .005 in. for flatness. Fig. 1 pictures the grinding of 4-in. slab units on a 30-in. double spindle grinder.

FLOOR AND ROOF SLABS

Strestcrete floor and roof slabs are assembled from elements having a surface area which measures 7½ in. by either 16 or 24 in. Thicknesses of 3, 4, 6, 8, 10, 12 and 16 in. are used as required for the designed loading and

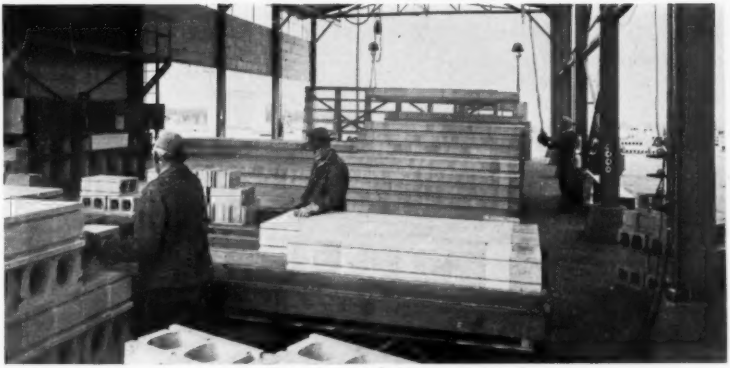

Fig. 2—Assembling 4 ft wide panels of 8 in. thick floor slabs

intended span. Planks, or panels composed of multiple planks, are assembled on a precision makeup table using a plurality of ground elements with steel plates at the ends of each assembly (Fig. 2). Threaded reinforcing rods are placed in splines molded into the sides of the units and laced through corresponding holes in the steel end plates. Nuts are placed on the ends of the reinforcing rods and tension, not exceeding the allowable working stress of the rods used, is applied by tightening the nuts with a torque wrench (Fig. 3) or by hydraulic jack.

Erection is normally handled by crane. Fig. 4 shows the placing of 3-in. roof slabs and Fig. 5 shows a 6-in. floor installation nearing completion. Panels average 4 ft in width but have been successfully handled in sections as large as 8 ft wide and 32 ft long. Form work is not required and erected panels are immediately available as working platforms for following craftsmen. A minimum of concrete grout is poured between slabs in the form of a joist for coverage of exposed steel and to key slabs together. Concrete topping may

Fig. 3—Prestressing reinforcing rods to pre-determined values in a panel of 10-in. thick floor slabs using a calibrated torque wrench

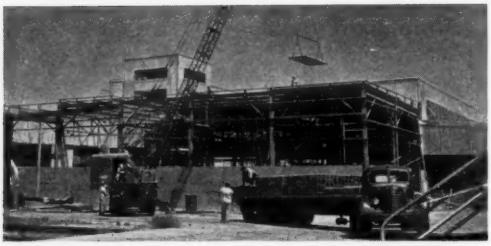

Fig. 4—Placing 3 in. thick panels of roof slabs on a steel frame building. Panels measure 4 x 8 ft and weigh 20 psf

Fig. 5—An installation of 6-in. Strestcrete floor slabs

or may not be placed as required for additional strength or wearing surface. Continuity of steel is obtained by bending reinforcing rods into the joist sections before grouting. This same open joist permits the placing of negative steel over supports to gain the added advantage of continuous beam design.

WALL SECTIONS

Tongue and groove siding

Of the various wall sections developed by the Basalt Rock Co., tongue and groove siding has been the most promising and has been adapted to varied types of installations. The material was originally intended for use as a non-load-bearing curtain wall for the exterior of steel frame buildings. Like the floor and roof slab, it is assembled from precast segments, but instead of having a spline, holes are provided through the elements to receive the reinforcement that consists of $\frac{3}{8}$- or $\frac{1}{2}$-in. diameter round steel threaded rods. A tongue is cast at one end of the unit with corresponding groove at the other end. Panels are assembled by pre-tensioning the reinforcing rod against

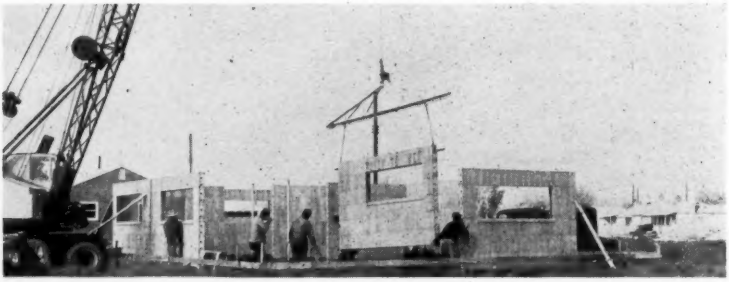

Fig. 6—Erecting wall panels of tongue and groove siding for a three-bedroom residence

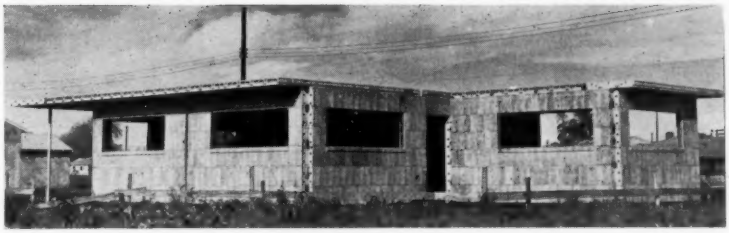

Fig. 7—Residence in Fig. 6, with 6 in. roof slabs in position

steel plates positioned at the end of each assembly of concrete segments. Mastic is used to seal the tongue and groove joint between individual planks of an assembled panel. The ground surface between units of each plank is sealed by the application of waterproofing paint after the material is erected. Attachment to steel frame buildings is accomplished by welding or bolting the end plates to the steel columns.

Recently, tongue and groove siding made its appearance in the low-cost housing field when an 1150 sq ft one-story residence was transported 60 miles from factory to jobsite and erected in three hours (Fig. 6). The roof, also of Strestcrete, was placed the same day (Fig. 7).

Another unusual application was demonstrated by constructing a two-story mill timber frame factory building with panels of the material bolted to the wood columns (Fig. 8).

Integral steel frames

Extensive use of tongue and groove siding led to the development of integral steel frame buildings. By substituting light steel channels for the flat end-plates and welding the channels of adjoining panels, load bearing box type steel columns are formed within the Strestcrete wall, Fig. 9. A fusion

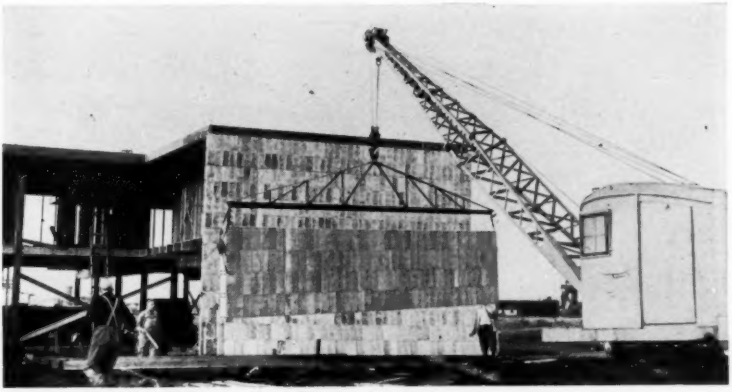

Fig. 8—Placing panels of 6 in. tongue and groove siding on a mill timber frame building

Fig. 9—Welding plate steel girder to columns of a commercial type integral steel frame building

welded steel plate girder of special design which is shown being welded, attaches to the columns and supports a Stresterete roof. This completes a standardized package building. Walls as high as 22 ft 8 in. and clear spans up to 56 ft are available. The size of pre-assembled sections is limited primarily by the capacity of equipment to transport and erect them. Completed buildings of this type are pictured in Fig. 10 and 11.

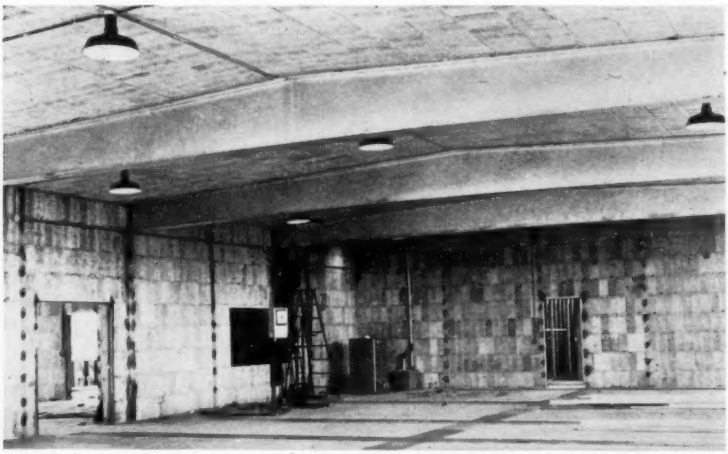

Fig. 10—Interior of a portion of an integral steel frame building showing plate steel girder and Strestcrete roof

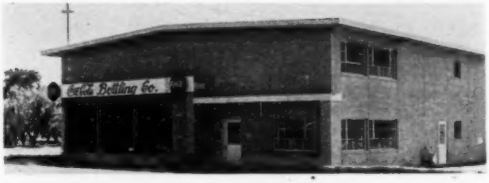

Fig. 11—Completed integral steel frame building with overhanging eave

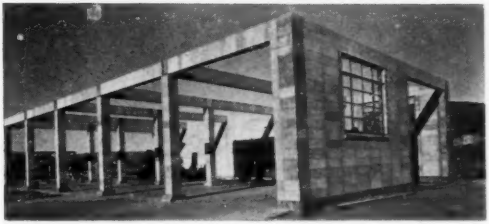

Fig. 12—Wall, columns and beam construction using Studwall

Studwall

Strestcrete Studwall is a form of mortarless block construction usually erected at the jobsite but occasionally pre-assembled into large panels at the factory and positioned by crane at the jobsite. Fig. 12 illustrates this type of construction with walls, columns, and beams erected without the use of mortar except for grout poured into the voids containing steel reinforcement. Units are a 24 in. long precision ground three-cell open end design. Vertical reinforcing rods with threads on one end are embedded in the concrete footing on 2-ft centers. When the blocks are stacked one upon another, cells align vertically with the open ends of adjoining units forming a cell around the reinforcing rod. After walls have been built to the desired height, clamps are placed over the rod and the entire assembly is bolted to the foundation. The cells containing the rods are then filled with concrete, thus reinforcing the wall and bonding the blocks together internally. Columns and beams are constructed in the same manner using standard as well as special shaped concrete blocks.

Columns and tanks

Fig. 13 illustrates the use of standard concrete masonry units for the con-

Fig. 13—24 x 36 in. H-columns constructed of precision ground concrete masonry units of conventional design. Reinforcing is prestressed

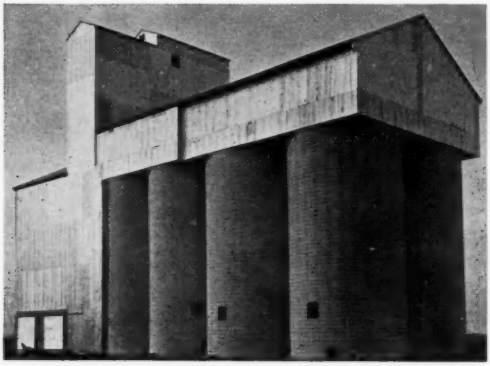

Fig. 14—12 and 18 ft diameter Strestcrete grain bins, 40 ft high constructed of 4 in. thick Studwall units

struction of 36 x 24-in. H-columns supporting the craneways for a 15-ton crane. Steel plates with holes corresponding to those in the block are inserted at intervals to serve as stirrups and take shear stresses. These also provide transverse reinforcing between units of the composite assembly. In this form of construction, all voids are filled with concrete after erection.

Grain bins, Fig. 14, silos and water tanks, are constructed in the same manner using radius Studwall units 4 in thick.

CONCLUSION

Strestcrete design calculations and assumptions do not differ from those for conventional concrete slabs. ACI T-Beam formulas are used. Unit stresses of concrete and steel are those normally used for concrete and steel of equal strength. The advantage gained in reducing deflection by pretensioning reinforcement is not considered in design calculations. Spans exceeding normal code limitations of ratio of thickness to span are not used except in experimental work. Loss of tension in reinforcement due to shrinkage of concrete has not been important when units have been properly cured. Creep of steel does not occur at the stresses used. Plastic flow deformations have been lower than that encountered for conventional monolithic design. While not yet fully substantiated by test, it appears that this may be partly due to the low water-cement ratio used in the mix of the machine-made elements.

It is not suggested that the products or method of construction described offer a solution to the many complicated problems inherent in precast concrete construction. It does, however, point out a course by which major concrete products manufacturers in various parts of the country are endeavoring to provide more economical components for the simpler type of buildings. Although relatively new in the present form, its greater speed with consequently lower cost has been demonstrated by many installations.

The use of precast structural elements reinforced with pre-tensioned steel is still in its infancy. Because of the number of individuals endeavoring to improve and develop practical applications, it is probable that the future will see an expanded use of construction of this type.

Tilt-up Construction in Western United States*

By F. THOMAS COLLINS†

SYNOPSIS

Precast concrete construction is increasing rapidly in the United States. The method described is that commonly known as "tilt-up" or flat cast construction which has proved most economical and most readily adopted by the general contractor in Southwestern and Western United States. This method makes possible savings in material and skilled labor which are becoming more critical under present conditions.

Data and recommended procedures are given for casting platforms, forming methods, bond breaking agents, wall panel design and construction, and erection of precast members. The patent situation affecting precast construction is discussed briefly.

INTRODUCTION

Tilt-up construction of buildings and other structures is now extensively used on the Pacific Coast, particularly in the Southwestern area. Since the general history of precast concrete construction has been well documented by previous articles in the ACI JOURNAL and other publications this paper is confined to work done in Southern California in the last five years and to current progress in this type of construction. Tilt-up construction incorporates the following principles:

1. Precast members are usually cast flat, resulting in flat slabs or rectangular cross sections for structural members. This is in contrast to the "ribbed thin-shell" technique advocated by Amirikian.‡
2. Members are cast on the site, the floor slab of the building usually being used for the casting platform. This differes from the usual practice of shop fabrication long used in Europe and the United States.
3. The heavy units are almost always erected by large 15- to 50-ton mobile truck cranes.

BRIEF HISTORY

The Los Angeles area is one of the most active in the United States in interest and actual use of this method of construction. Latisteel, Inc., was one of the pioneers in tilt-up wall panels for buildings. Latisteel construction is not true tilt-up in that the wall panels are essentially steel frames with a thin concrete cladding but the erection methods are similar. In 1945, Latisteel erected the J. H. Davies building in Long Beach by this method. Wall panels

*Presented at the ACI 47th annual convention, San Francisco, Calif., Feb. 21, 1951. Title No. 48-10 is a part of the copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 2, Oct. 1951, *Proceedings* V. 48. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Feb. 1, 1952. Address 18263 W. McNichols Rd., Detroit 19, Mich.

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‡Amirikian, A., "Precast Concrete Structures," ACI JOURNAL, Dec. 1946, *Proc.* V. 43, p. 365.

were cast on the Truscon decking as a casting platform. The panels were stacked in layers of three or four panels because of the constricted casting site. In 1946 the same firm constructed the Hoagland Grocery Co. building in Long Beach. Wall panels, cast on Truscon steel decking laid on the concrete floor, were erected with large motor cranes.

In waterfront structures such as wharves, warehouses, piling, etc., considerable use has been made of precast concrete construction by several large Pacific Coast construction organizations. Two of the most outstanding are the firm of M. H. Golden of San Diego which developed a method of precasting concrete piling for wharves and waterfront work as early as 1941 and the contracting firm of Swinerton and Walberg of San Francisco which was engaged in similar efforts with Ben Gerwick at about the same time.

The W. P. Neil Co. of Los Angeles has been very active in tilt-up construction since 1941 using this method extensively in the construction of numerous Navy ammunition warehouses at Hawthorne, Nev. This company developed the "Neil System" of tilt-up, used by Daly Bros. of San Francisco and now by the National Engineering Corp. with headquarters in Palo Alto, and is presently actively engaged in research on tilt-up wall panels designed as unreinforced masonry and other research on prestressed roof members. One project now under construction by this firm is the Chrysler missile plant in East Los Angeles. The walls of this $5,000,000 project are all precast tilt-up panels.

Fig. 1 and 2 indicate the growth of this method on the Pacific Coast since 1946. Fig. 1 shows the increase in number of contractors and Fig. 2 shows the growth of the work of two individual contractors who adopted this method of construction.

PATENTS

Many patent applications on systems of precast concrete framing, methods, materials and designs relating to precast concrete construction have been made to the United States Patent Office; some of which have been granted. Examples are (1) Halverson-Treichel patent No. 1,326,400 (now expired), (2) E. J. Rappoli patent No. 2,531,990, issued Nov. 28, 1950 and (3) Patent No. 2,531,576 issued on the same date.

Present patent situation

The Halverson-Triechel patent issued in 1919 has now expired so that it is in the public domain. One of its claims read:

> The method of constructing the walls of reinforced concrete buildings consisting of forming the walls on the floor of the building by using the floor as one side of the mold and then by limiting the size of each wall by an edging of planking temporarily secured to the floor, the height of such edging being the thickness of the wall.

The E. J. Rappoli patent covers the making of wall slabs in horizontal molds stacked one on top of another. The bond-breaking agent was a lacquer. The use of molds and an expensive bond-breaking agent makes this patent unimportant to true tilt-up methods requiring no molds and using a very inexpensive bond-breaking agent.

Fig. 1 (right)—Increase in number of building contractors doing tilt-up construction on the Pacific Coast

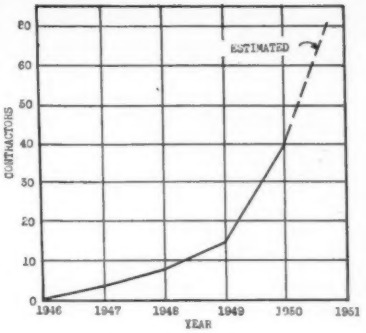
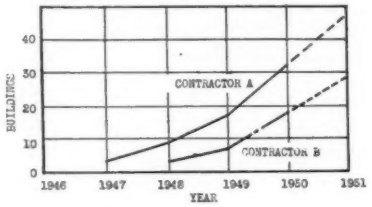


Fig. 2—Growth of tilt-up construction by two contractors



The third patent, No. 2,531,576, creates more of a problem than the first two mentioned. The patent makes one broad claim:

> The method of making a concrete building member comprising, casting a concrete base layer, sealing the exposed top surface of said layer by coating with a liquid sealing composition containing a substantially water insoluble film forming substance in an evaporable liquid vehicle, which substance upon removal of the liquid vehicle will deposit on the concrete layer as a uniform, substantially water impermeable and easily separable sealing film which will prevent formation of a bond between the concrete base layer and the concrete poured wet upon such a deposited sealing film and set and cured, thereon, casting a wet concrete layer on the exposed coated surface of the base layer after said base layer has at least partially set, thereby producing a superposed concrete layer on the coated and at least partially set base layer, the layers being separated by said film, permitting the superposed concrete layer to set and cure in its position on the sealing film until the thus produced building member has attained sufficient strength to be lifted free of the base layer, and then removing the produced building member from the base layer.

This claim describes the fundamentals of tilt-up construction or any other economical and practical precast concrete construction. It describes in very general terms the economical and efficient bond-breaking agent. The description is so broad that it seems to cover any similar material without naming any specific ingredient in the agent. It also claims the method fundamental to all precasting and in use for well over 30 years.

If this patent is upheld by the courts then it is conceivable that a general contractor, unless he is well advised and has knowledge of other bond-breaking agents, will be faced with paying royalty to continue precasting operations. Every contractor and builder knows the value and economy of a membrane curing agent for curing concrete; nearly all floor slabs are now so cured. It is possible that the builder might be restrained from using a membrane curing agent on the floor slab if he was intending to precast unless he paid a royalty. The validity of this patent should be tested as soon as possible to clarify the contractor's position in regard to it.

CASTING PLATFORMS

To precast a concrete unit, one must have a casting platform which for

flat sided members must be a plane surface. The plane may be out of level but it must be relatively smooth. Casting platforms used in the past and in current use are:

1. Earth—should be fairly smooth, compacted and hard.
2. Concrete slab—usually the floor slab or auxiliary concreted area.
3. Wood planks—not recommended; form marks show.
4. Plywood—sheeting built on a platform; an emergency measure.
5. Casting table—used in commercial precast factories or yards.
6. Wood molds—required for ribbed sections of minimum quantity.
7. Concrete molds—expensive and require maximum re-use.

Earth is an economical and practical casting platform. Weeds and organic matter must be removed as well as top irregularities and dust. Dust or sand tends to billow out and cause the concrete to bunch into knobs. For a dock-high warehouse, the lower panels can be cast on earth with the top finished side being the exposed exterior face of the wall. The under or dirt side contacts the fill.

The floor slab of the building itself is the next most economical, the most conveniently located and involves little additional cost. When there is insufficient room on the floor slab to cast all the units and erect them in proper sequence an auxiliary casting platform is necessary. This is usually most economically done by placing nearby a 2-in. stiff-mix concrete slab and finishing it smooth. When no longer needed it can be broken up and sold as paving stones or garden flag stones.

Slab requirements

Building floor slabs used as casting platforms must be built with this use in mind. Since the slab must support the heavy concentrated loads of the motorized crane used in erecting the heavy precast units the following fundamentals are essential:

1. Fill must be properly consolidated to insure a compact subgrade. The compaction of the subgrade is more important than any other one thing in making floor slab casting platforms.
2. The floor slab should be at least 5 to 6 in. thick. A good average is 5½ in. which allows use of 2 x 6-in. screeds.
3. High-strength concrete should be used; 3000 psi concrete is preferred. The concrete should be placed so dry that it must be vibrated into place. Slumps of 3 to 3½ in. are recommended.
4. The use of steel mesh in the floor slab is a waste of steel in the opinion of the author. If steel is required then ¼- to ⅜-in. bars should be used. The recommended position of the steel, if used, is in the middle of the slab.
5. A screed type vibrator is recommended.
6. If a floor hardener is used the method of application should not conflict with spraying the bond breaking agent on the floor slab.
7. Large aggregate, 2½- to 3-in. maximum, can be used to advantage in the floor slab.

FORMING METHODS

One of the great advantages of precast concrete construction is the great saving in forming labor and material compared to cast-in-place concrete. Some precast shapes require less forming than others; solid wall panels, for

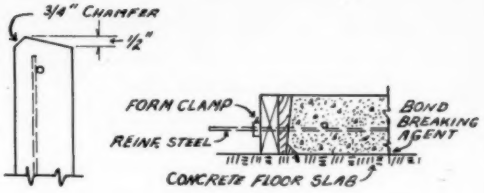


Fig. 3—Parapet design and method of forming

example, require forming only of the periphery of the wall panel since the two flat sides require no forms. As precast shapes become more complicated more forming is required. It may become so complicated that, even with precast construction, there is no net saving when the erection and joining costs are considered.

Thickness of wall panels

Since 90 percent of precast concrete work will probably continue to be building wall panels, forming them is first considered.

The minimum thickness of a reinforced concrete wall recognized by the Bureau of Fire Underwriters for fire rating is 6 in. The Bureau rates a 6-in. wall as a 4-hour wall. The minimum practical thickness recognized by Building Departments and building codes as having any fire rating is a 4-in. wall. A 4-in. wall is rated as a 2-hour wall by most building codes; a 2-in. wall is rated as one hour but is impractical for higher tilt-up walls.

There is little difference between the cost of a 4-in. and a 6-in. tilt-up wall. The minimum practical thickness for commercial and industrial construction, as far as the author is concerned, is $4\frac{3}{8}$ in. However, there is no great saving in a wall thinner than $5\frac{1}{2}$ in. A wall formed with 2 x 6-in. material with a net width of $5\frac{5}{8}$ in. will usually give a net thickness of 6 in. because of unevenness of the floor slab and the tendency of the concrete to screed higher than the form.

Wall panel forming

Parapet and water table—Recommended design of the parapet is shown in Fig. 3. The method of forming is shown at the right. The two details, for top of parapet and water table or bottom of wall panel, will probably be the same in most walls. The top of the parapet is sloped toward the roof at the start of the $\frac{3}{4}$-in. chamfer to allow drainage toward the roof. This keeps dirt from streaking the face of the wall when it rains. It also allows the parapet flashing to be brought right to the edge of the chamfer insuring good sealing of the roofing.

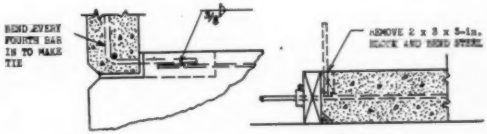


Fig. 4—Water table and forming method

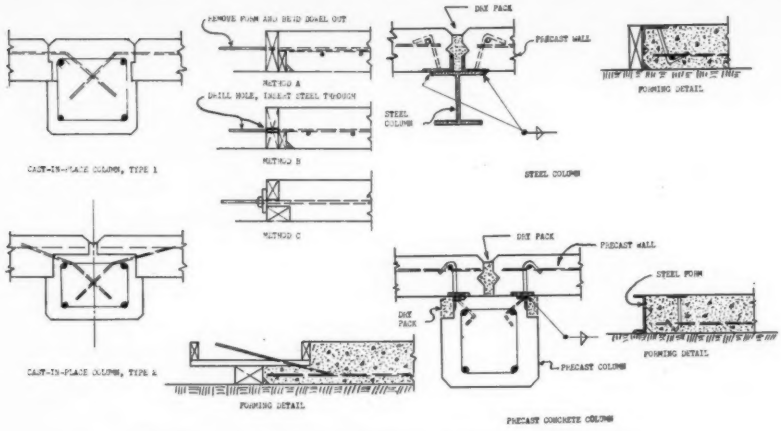


Fig. 5—Typical column designs and method of forming panels

Vertical edges—Forming required for the vertical edges of the wall panel will depend on the design of the structure (Fig. 4). Four main types of column design are shown in Fig. 5. Forming for each type is at the right.

BOND-BREAKING AGENTS

The most important single item in tilt-up construction is the bond-breaking agent. Throughout the development of precast concrete construction there has been a continual search for more economical bond-breaking agents. Some of the early materials used, and in some cases still being used, are sand, oil, grease, lard, chalk, stearic acid, castor oil, fuel oil and lacquer.

Modern bond breaking agents, in order of practicability and economy, are:

1. Hunt's Process solution Bx 112-TU, Techkote tilt-up compound
2. Hunt's Process curing solution 130, Techkote anti-bond No. 200
3. Paraffin dissolved in gasoline
4. Wax dissolved in petroleum ether or gasoline
5. Green liquid soap
6. Refined commercial pure soap in powder form dissolved in water
7. Blend of castor oil and fuel oil
8. Talcum powder
9. Ground chalk dissolved in water and applied as paint
10. Water glass
11. Paper (Sisalkraft or equivalent)
12. Canvas
13. Rubber or plastic sheeting

The author prefers the bond-breaking agents listed under Item 1 above. There are no doubt equivalent materials on the market. The following procedure is outlined for its use and probably applies to similar agents:

1. After the final steel troweling of the floor slab apply one curing coat of Bx 112-TU or Techkote tilt-up compound with a hand sprayer fitted with a recommended noz-

zle. Adjust the spray to a rather fine mist and make the strokes long and sweeping with some overlapping. The technique resembles that for spraying lacquers. This first coat is usually applied at the end of the day, covering about 250 sq ft per gallon. The next morning the carpenters can lay out and form the wall panels.

2. As soon as all wall panel forms are completed and before placing any steel or steel inserts sweep or blow all debris from the casting platform. Then spray another coat of the bonding agent at about the same or slightly lighter intensity than the curing coat. Spray wood forms also as it will help to seal them from moisture in the concrete and will facilitate their removal.

3. The casting surface should not be sticky or slippery. Sprinkle a few drops of water on the surface. If the drops form into globules and are not absorbed by the concrete as a blotter would absorb ink there is ample material on the slab for bond breaking.

4. Place reinforcing steel followed by the concrete which is vibrated, screeded and finished in the customary manner. After the final troweling apply a curing coat of Bx 112-TU or Techkote tilt-up compound in the same way as for the floor slab.

WALL PANEL DESIGN AND CONSTRUCTION

Preliminary design considerations

The question has often been asked, "When should the designer make the wall panel in one piece from the parapet to the bottom of the grade beam, and when should he break it into a lower and an upper panel?" Each job is best decided on its own merits taking into consideration:

1. Capacity of motor cranes available for erection.
2. Height from the finished floor to the bottom of the lower chord of the truss.
3. Type of fill available if it is a dock-high job requiring extensive fill under the floor slab.
4. Available casting and craning areas and space for maneuvering the truck crane.

As a general rule, for dock-high buildings with 6-in. walls and ceiling height of 14 ft or more, the advantages lie with making the wall in two panels. Since this is a common type of building, methods of casting these panels are described.

Lower dock-high panels

The lower panels of a dock-high building may be cast separately from the main wall panels using as a casting platform the central part of the floor slab already placed on the fill as shown in Fig. 6.

Under some conditions, it may be necessary to cast the lower panels on the ground. The design of the lower panel is improved in many cases if there is a grade beam of increased thickness at the bottom. This allows greater bearing area and increases the ability of the panel to span from footing to footing. Fig. 7 shows the casting position for this type of panel.

Steel that is to extend into the floor slab when the lower panel is erected in the building frame can either stick out of the top of the form, with a removable block as shown in detail A, Fig. 7, or it can be bent down into the

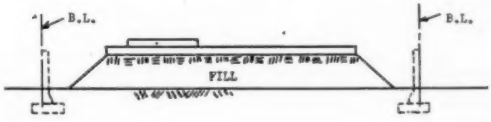


Fig. 6—Floor on fill used as casting platform

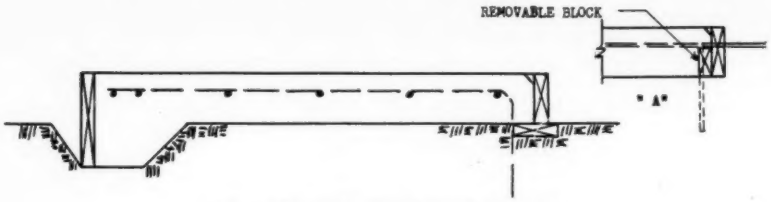


Fig. 7—Casting lower wall panel on ground

ground. Bending the steel into the ground aids in placing and holding the steel. This positions the steel correctly in relation to the main floor slab.

Forming for panel edges is shown in Fig. 8. The blocks placed in the top of the wall are merely a construction aid to allow the temporary nailer to serve as a screed for the placing of the floor slab and then to serve as a nailer for securing the forms for the upper wall panel.

ERECTION OF PRECAST CONCRETE MEMBERS

Truck crane

The most practical way to erect most tilt-up buildings is to use a modern truck crane of sufficient lifting capacity to handle the precast units involved. For most tilt-up work the capacity should be at least 20 tons.

Rates for truck cranes

In Los Angeles, rates for crane service are standardized by the crane association. These rates are:

Capacity, tons	*Rate per hour*	*Overtime*
15	$13.80	None on crane
20	15.80	None on crane
35-45	17.00-18.00	None on crane

These rates include the operator and his assistant. No overtime is charged for the crane but overtime is charged for the operators. If they are working with crafts, the regular double time overtime pay is charged. If the overtime is travel time, the charge is for time and a half.

Summary of erection costs

The hourly cost of equipment and labor for a typical tilt-up erection job is about 75 cents per minute. With this cost going on every minute it is important to streamline the operation to keep erection costs at a minimum.

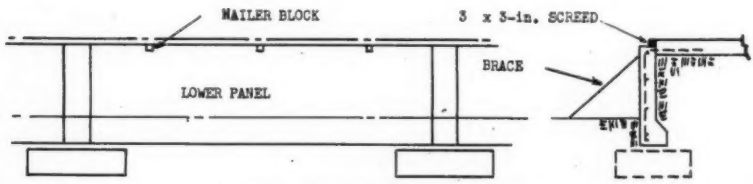


Fig. 8—Details of top panel edge

Motor crane with two operators	$17.00
Two riggers @ $2.60	5.20
4 carpenters @ $2.35	9.40
2 laborers @ $1.75	3.50
1 concrete man @ $2.50	2.50
1 foreman @ $3.25	3.25
Miscellaneous equipment rental	4.00
Total cost per hour	$44.85

Packaged erection service

It is evident now, after five years of extensive tilt-up construction in Los Angeles, that the ideal way for all concerned is a "packaged" erection service. Equipment contractors should have all the equipment necessary to erect and brace the precast units. It is slowly coming to that in Los Angeles and contractors elsewhere beginning tilt-up operations will benefit from this experience. Despite additional rental for the erection equipment, if it is once standardized for each area, the over-all cost of tilt-up construction can be further reduced.

Pick-up method

Since most precast concrete members will be erected by point pick-up this method will be discussed in detail. First consider a wall slab 19 ft high, 19 ft wide and 6 in. thick. Each square foot of the slab weighs 75 lb or a total slab weight of 27,175 lb. Let us consider two ways of lifting this wall panel; by attaching to the top edge and lifting, and by pick-up points 5 ft 6 in. down from the top.

Lifting from top—Lifting from three points at the top of the wall we find the moment caused by the slab bending between the lifting point and the floor to be about 21.3 ft-kips per lift point or 6 ft 4 in. width of wall. Concrete stresses are about one and a half times what would be allowed and the steel required would be ½-in. rounds at 6-in. spacing, at 2 in. clear from the bottom edge.

Intermediate lifting points—By lifting at a point 5 ft 6 in. down from the top, the moment from the overhang tends to subtract from the moment at the middle of the sagging portion between the pick-up point and the floor. The sagging, or positive, moment is M_c and is 7.85 ft-kips compared to the 21.3 ft-kips when lifted from the top. The negative moment is about the same, or 7.15 ft-kips. Steel required to reinforce for the negative moment is 4½-in. bars 6 ft long directly over the pick-up points. The regular panel steel takes care of the positive moment.

Comparison—To lift the panel by the top would require 175 lb of additional steel costing about $24 in place. Lifting by point pick-up the additional reinforcing steel over that required by code would be 48 lb costing $6.70. Apportioning the costs this would be 1¾ cents per sq ft. On a wall area of 20,000 sq ft the difference in the cost of the building would be $900.

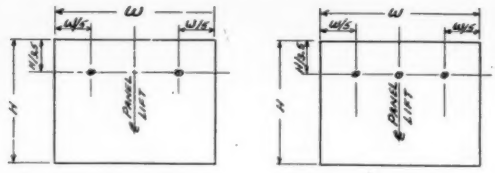


Fig. 9—Spacing of pick-up points

The above comparison was made assuming it was possible to lift the panel by the first method. However, to so lift it would probably crack the panel so badly it could not be used.

Two point versus three point pick-up

Many of the early tilt-up buildings in Los Angeles showed erection cracks which were partly attributed to the two point pick-up system then in general use. Now erection is about evenly divided between two point and three point pick-up. The latter provides additional assurance against cracking the panel during erection but the question arises as to when the two methods should be used. For practical purposes, the limiting width of 6-in. panels for two point pick-up is about 17 ft. For a building with truss spacing close to 18 ft two point pick-up may be used. The rule of thumb spacing for pick-up points, without strong backing, is as shown in Fig. 9.

Low walls—Walls less than 10 ft high can be picked up from the top. The pick-up may be a hair-pin bar out of the top of the panel or a pick-up insert. For dock-high walls the usual procedure is to use the hair-pin pick-up.

Bracing equipment

It is general practice to brace precast wall panels, columns and bents during erection. Wall braces should be designed for general use in bracing all other precast members except the lower panels for dock-high retaining walls.

Lightweight tubular turnbuckle wall brace—This wall brace is made from seamless steel tubing, with left and right hand threaded nuts at opposite ends. Right and left hand threaded bolt stock attached to the end shoes allows lengthening and shortening the brace by turning the tubular central portion of the brace (Fig. 10). This brace is designed to withstand a 15 psf wind load on a 20 x 20-ft wall panel using one brace at the center of the panel.

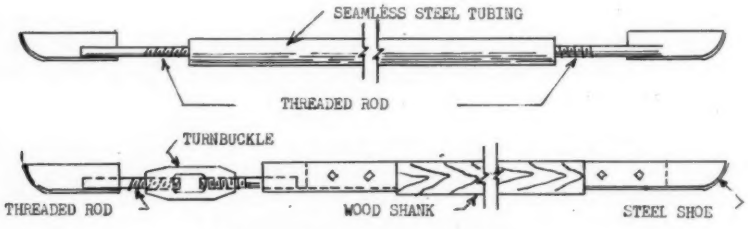


Fig. 10—Common types of wall braces

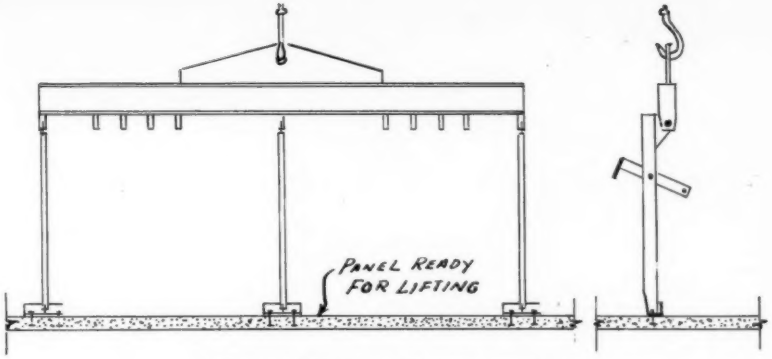


Fig. 11—Lifting beam with stiffleg attachment

Wood brace with turnbuckle on one end—This is the typical wall brace generally used in precast tilt-up construction with some improvements (Fig. 10). It requires two men to install and is critical at the turnbuckle for compression loads. It is unsafe to use this brace alone on a 20 x 20-ft wall panel. Its only advantage is its lower first cost.

Flexible cable versus stiffleg attachment

A stiffleg attachment (Fig. 11) to the pick-up points is recommended for erecting wall panels. This attachment allows the use of a dog which can be operated from the ground. The cable attachment is sometimes used with a dog that catches the top of the panel but it must be hand set by a rigger climbing the panel.

Pick-up inserts

Many types of pick-up inserts have been tried and discarded. The two best now in general use are shown in Fig. 12 and 13. Some fundamental rules for pick-up inserts are:

1. Standard U. S. extra heavy nuts, preferably square, should be used for threaded attachments. Minimum size is 1 in. for three point pick-up and 1⅛ in. for two point pick-up, with 1¼ in. preferred.

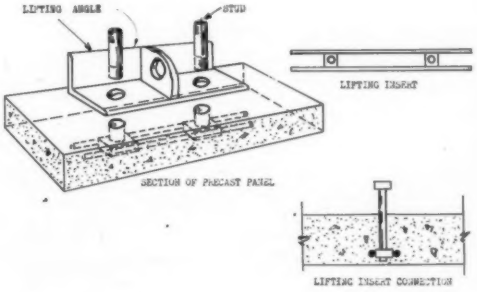


Fig. 12—Lifting angle and inserts

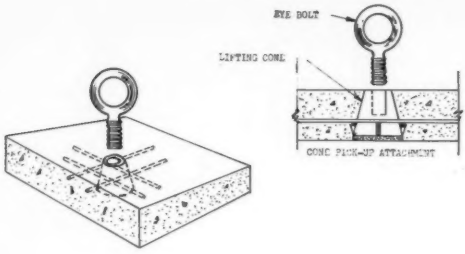


Fig. 13—Lifting eye and cone insert

2. Inserts should be simple and economical to make.
3. Reinforcing steel should be deformed and of intermediate or structural grade.
4. Use built-in chairs and sleeves to save time and trouble.
5. If an attaching angle is used, use studs instead of bolts to engage insert nuts in the concrete member. Extend stud ½ in. through the nut, and apply a heavy coat of grease to the extended end.

Wall brace anchors

A brace should also act as a plumbing device and should be capable of taking compression as well as tension loads. This means that both ends of the brace must be anchored. It is important that anchors be capable of taking the design load of the brace. The following rules apply when using only one brace per panel, at its center.

1. Minimum size of nut recommended is ¾ in.
2. Use studs instead of bolts.
3. Long coupling nuts are recommended instead of Standard U. S. nuts.
4. Reinforcing steel welded to the nut should be deformed, at least ½-in. round and should be at least 18 in. long.

Position of motor crane

There is a definite advantage in speed of erection in craning from the floor slab. Sometimes it is necessary to position the crane outside the building, but soft subgrade may endanger its stability. Craning from outside the building usually requires that the outside of panels be cast up. This is a disadvantage except for painting. For speed in erection:

1. Lay out casting for minimum handling of precast units.
2. Cast outside wall panels face down.
3. Set pick-up inserts from top inside face.
4. Use studs instead of bolts.
5. Keep vertical steel 2 in. clear of floor slab.
6. Plan erection for craning from floor slab whenever possible.
7. Use properly designed erection equipment and wall braces.

CONCLUSION

From the foregoing it is evident that success in tilt-up construction depends on thorough and minute planning, attention to many details and the efficient use of erection equipment. The ability to improvise and develop new methods to improve the quality of his work is an important asset to the contractor in this field.

Discussion of a paper by F. Thomas Collins:

Tilt-Up Construction in Western United States*

By SAMUEL HOBBS and AUTHOR

By SAMUEL HOBBS†

Mr. Collins has presented a brief outline which well describes methods and practices which have been typical of tilt-up construction in southern California. His experience as a pioneer in the development of this type of construction is such that a wealth of detail concerning the solution of problems in designing, forming, erecting and connecting precast sections has necessarily been omitted from his summary of the subject.

Integration of the entire structure to resist lateral as well as vertical forces is a prerequisite for the design of buildings in areas subject to earthquakes. The methods used in making connections and their proper execution are vital to successful use in such areas. Good connections, well detailed and properly installed, will produce the desired integrity of structure. Moreover, the integrity of such a structure is not limited merely to the capacity to resist the forces on which design is predicated.

For example, a disastrous fire occurred on Dec. 9, 1951, in a Los Angeles factory and office building, burning with great intensity for several hours within the factory area. It resulted in complete loss of the roof structure of the factory and its whole interior, heavily stocked with material for use in manufacture. The non-fireproof girders and interior columns collapsed in a mass of crumpled ruins under the intense heat. In spite of the strain of this collapse and of the severing of roof anchorages, the precast walls with their cast-in-place columns remained stable, though affected by some scaling of exposed surfaces. The 5½-in. tilt-up fire-wall protected the office portion of the building adequately, confining office damage to a minor amount caused by water. The concrete walls were made with the usual sand-gravel aggregates, which would be classed as siliceous.

This case is cited because only in recent years has tilt-up construction become common for such buildings, and because this was a real "trial by fire" of structural stability. It is a matter of interest that the reconstruction, as far as the walls are concerned, will involve mainly a re-coating of the exposed surfaces with a moderate depth of shotcrete, the provision of proper anchorage for a new roof structure, and the installation of new sash.

*ACI JOURNAL, Oct. 1951, p. 133. Disc. 48-10 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE. V. 24, No. 4, Dec. 1952, Part 2, *Proceedings* V. 48.

†Engineer of Structures, Portland Cement Assn., Los Angeles, Calif.

It may appear that some of the foregoing comments are in the nature of a digression from the subject. To the writer, however, they seem pertinent because there have been critics who were prone to doubt that tilt-up methods would produce this kind of structural integrity.

In conclusion, it is to be hoped that Mr. Collins' excellent paper may lead to further development in the literature of the details of design and construction necessary to extend the usefulness and broaden the field of precast concrete construction.

AUTHOR'S CLOSURE

Mr. Hobbs has brought up two important points which should be given particular attention in precast structural concrete design and construction—integration of the entire structure, and adequate fire resistance.

The author is in accord with Mr. Hobbs concerning the importance of providing enough inert structural resistance to make the walls self-sustaining in the event of fire. In a Type III building there is a possibility of the roof structure collapsing during a fire, and if the walls are wholly dependent upon the roof system for stability, then they, in turn, may collapse.

Mr. Hobbs points out one system of building integration which has undergone a fire, in which the walls are essentially held intact. There are, however, a number of framing methods which would serve adequately well.

Many engineers now designing precast concrete structures are of the opinion that cast-in-place pilasters and other joints—vertical, horizontal, or otherwise, with adequate lap of dowels and reinforcing steel—are sufficient to tie the structure together. The big objection to this method, alone, is the development of cracks in the wall panels, and even the cast-in-place joints, as the concrete shrinks. It is the author's opinion that we must go beyond this method and use prestressed wire units in the precast members and tie the precast units together.

Wall panels can be prestressed diagonally or vertically for the precast unit itself, and when erected into place, all the members can be prestressed together with an adequate unit—say, at the roof line and the floor line (for one-story structures) or at the center of the panel, horizontally. The Freyssinet system of prestressing has distinct advantages for prestressing the structure for over-all integration. The Prestress Corp. and Roebling methods have certain advantages for prestressing the precast units individually. The Stressteel high alloy bar offers advantages and economy for vertical prestressing of the wall panels. Once there is compression on the joint, waterproofing is greatly simplified. For engineering calculations, a joint under compression will be allowed more shear value.

As regards fire resistance, there is a great deal to be said on this subject. The western flat cast tilt-up type of precast structural concrete construction has, by its very nature, more fire resistance than thin shell "ribbed" members. In the city of Los Angeles, the required thickness of a precast concrete wall for a 4-hr. fire rating has been increased from 6 to 8 in. Yet, other agencies

and ACI Committee 324 on precast concrete approve the 1¼-in. thickness of thin shell designs without any apparent question or consideration for fire rating.

The precast sandwich panels recently tested for the Vermiculite Institute offer a great many advantages for the flat cast western tilt-up method, especially in increased fire rating. With 2-in. outer shells of regular high strength concrete, and an inner core of vermiculite concrete varying from 2 to 4 in., the fire resistance is greatly increased. Other advantages are the increase in resistance to thermal conductivity, or lower *U* factor, and a lighter section. When this sandwich panel is prestressed, thus keeping the two outer shells in compression, the precast unit becomes an entirely new and different unit. It is so promising that a great deal of research should be done upon it. So far, the research has been limited to a few private engineers.

Precast structural concrete construction has grown from practically nothing in 1945 to an estimated one billion dollar business for 1953. The construction industry is now the number one industry in the United States, as far as gross national product. Methods of construction are now undergoing a revolution, long needed, but with still a long way to go. It seems only fair, reasonable, and sound thinking that more attention be given to research and development by private consulting firms, financed by federal or state funds, on precast concrete construction methods and design. Research and development would result in lower costs, better buildings and material savings so badly needed at this time.

Diagonal Tension in Reinforced Concrete Beams*

By ARTHUR P. CLARK†

SYNOPSIS

Resistance of reinforced concrete beams to diagonal tension was investigated in a series of tests which included beams with no web reinforcement and beams with varying ratios of web reinforcement consisting of ⅜-in. round deformed bars. Strains in the web and tensile reinforcement were measured with resistance strain gages; deflections of the beams under load were also measured and the number and extent of cracks were observed.

Beams of two cross sections, four span lengths and concrete strengths ranging from 2000 to 6000 psi were tested for five different positions of concentrated loads. One of the results of these tests, not previously demonstrated, is that the position of the loads on a beam influences considerably its shear carrying capacity. An empirical formula based on the data obtained in this study indicates that the shear resistance of the beams varies as the square root of the percentage of web reinforcement and linearly as the compressive strength of the concrete multiplied by a factor representing the ratio of effective depth of beam to distance from plane of the nearest concentrated load point to plane of the support.

INTRODUCTION

Beams of reinforced concrete, more than those of metal, have a potential weakness in the region where loads are transferred to the supports through the web. Vertical stress caused by the end shear combines with longitudinal stress from bending in the beam and produces tensile and compressive stress components. The tensile component is the critical one as concrete while strong in compression is comparatively weak in resisting tension. This tensile component is diagonal tension. Since the end shear is used as a measure of its magnitude, diagonal tension is commonly called shearing stress. Reinforcing steel, generally in the form of stirrups, is used to resist this tension.

A theoretical analysis of the stresses in the web of a reinforced concrete beam is at best only approximate, and it becomes necessary to rely to a considerable extent on the results of tests for a determination of the resistance of a beam for varying concrete strengths, ratios of web reinforcement and other factors. Design of web reinforcement at present is based on semi-rational methods, and there is need for additional test data to evaluate the factors which influence the resistance of a beam to diagonal tension and for a general expression which will indicate the shear capacity of any reinforced concrete beam.

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†Member American Concrete Institute, Research Associate, Fellowship of American Iron and Steel Institute at the National Bureau of Standards, Washington, D. C.

It is known that the capacity of a reinforced concrete beam to resist failure by diagonal tension is influenced by a number of factors:

1. Compressive strength of concrete.
2. Amount, distribution and yield strength of web reinforcement.
3. Size, number and yield strength of bars used as tensile reinforcement.
4. Ratio of effective depth of beam to shear span (distance from the plane of the nearest concentrated load point to plane of support).

The investigation conducted in the National Bureau of Standards by the Research Fellowship of the Committee on Reinforced Concrete Research of the American Iron and Steel Institute was intended to produce additional information as to the action of shearing stresses set up in the web of reinforced concrete beams. Tests by previous investigators have usually been made on beams of one size under one type of loading. In this investigation the ratio of depth of beam to shear span and the ratio of web reinforcement were principal variables, and the effect of varying the amount of tensile reinforcement and the strength of concrete was also studied. Strains in the reinforcing bars, the deflection of the beam, the pattern and extent of cracks and the ultimate load were recorded.

TEST SPECIMENS

The majority of the test specimens were beams 8 x 18-in. in cross section with a span of 6 ft and were made of a cement, sand and gravel mix proportioned to produce a concrete having a compressive strength of 3500 psi. There were in general three duplicate specimens for each variation of conditions. A few beams made with concrete having a compressive strength of 2000 psi and a few of 6000 psi were tested as well as a small number of beams having cross sections of 6 x 15-in. with spans of 8 to 10 ft.

The web reinforcement consisted of $\frac{3}{8}$-in. deformed bars. The main tensile reinforcement in the bottom of beams was straight round deformed bars with special anchorage consisting of $\frac{1}{2}$ x 8-in. steel plates $\frac{1}{4}$ in. thick welded to the ends of the bars to avoid possible failure by bond. Tests were

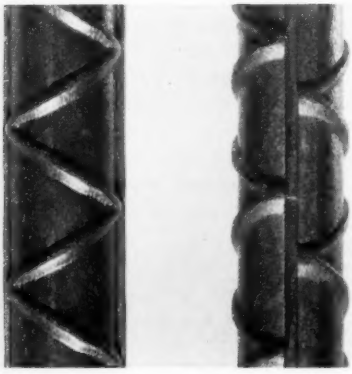

Fig. 1—Type of deformed bar used in tests

TABLE 1—PHYSICAL PROPERTIES OF BARS

Bar No.	Area, sq in.	Yield point, psi	Tensile strength, psi	Deformations		
				Spacing, in.	Height, in.	Gap, in.
3	0.111	48,020	70,880	0.271	0.032	0.08
7	0.599	53,710	86,900	0.581	0.056	0.12
9	1.012	48,630	72,840	0.862	0.065	0.12
10	1.267	46,500	76,830	0.921	0.074	0.13

started in 1948 and bars met requirements of ASTM Specification A305-47T then in effect. Fig. 1 illustrates the type of deformations and the physical properties of the bars are given in Table 1. Three or more tests were made on each size of bar.

Beam specimens were cast in steel molds and removed from molds after five days, then kept moist until the day prior to testing at 28 to 30 days age. Four or more standard 6 x 12-in. control cylinders of each mix, used in not more than two beams, were cast and stored in the same manner as the beams and were tested at the same age as the corresponding beam specimen to determine the compressive strength of the concrete.

Strains in the reinforcing bars were measured with SR-4 electric strain gages. The gages were attached to the longitudinal reinforcing bars at the center of the span and to both legs of the stirrups, in most specimens, at points where a diagonal line from the load point of the beam to the support crossed the stirrup leg. Deflection at the center of the span was measured by dial gages on each side of the beams; the dial gages were supported on aluminum bars whose ends were attached to the sides of the beam over the supports. The gages bore on angle clips attached to the concrete by screws.

TESTING PROCEDURE

Beams were tested in a 600,000-lb capacity hydraulic testing machine with the beam supported on rockers. The load was applied, other than for center loading, through a steel loading beam with a roller at one load point and a spherical bearing unit at the other load point. Steel plates 3½ in. wide by 1 in. thick were used to distribute loads and support reactions. A beam in the testing machine is shown in Fig. 2.

Five different positions of load application were used; center loading, two loads placed 6 in. each side of center, third point and quarter point loadings, and for the beam on a 9 ft 7 in. span the loads were applied 2 ft 8½ in. each side of center. Fig. 3 gives details of the beams and loading conditions. The beam was preloaded to about 500 lb and after removal of this preload the gages were read for zero loading. The load was then applied in steps of from 3,000 to 10,000 lb and at each step all gages were read and cracks traced and recorded. Loading was continued until failure except in the case of a few

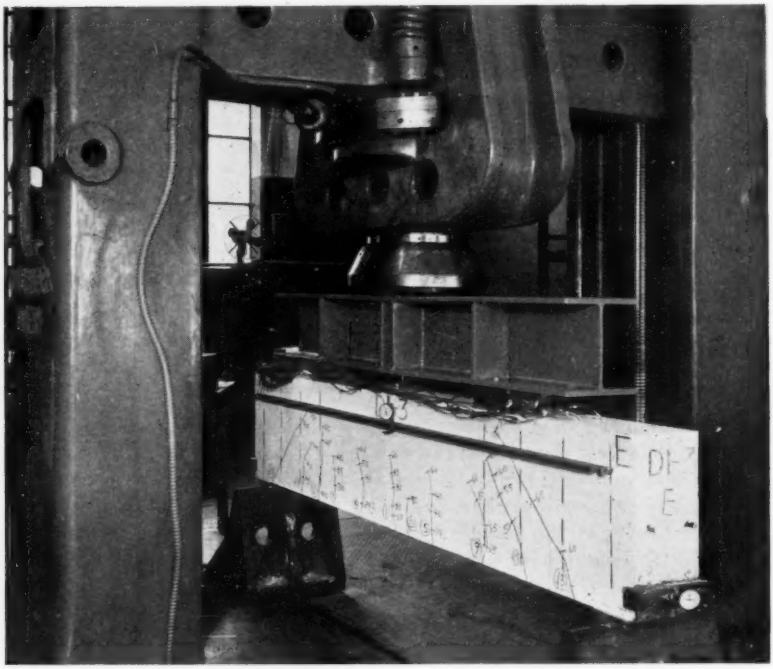

Fig. 2—Specimen in testing machine

beams where the loading had to be discontinued because of distress in the loading beam. All beams were designed for failure in diagonal tension, but in some cases yielding of the main reinforcing bars or compressive failure of the concrete occurring at about the time of diagonal tension failure made the primary cause of failure questionable.

RESULTS AND DISCUSSION

Standard notation is used in the following discussion and additional terms used are defined below.

A_v = area of two legs of a stirrup
a = distance from plane of the nearest concentrated load point to plane of the support
h = depth of beam
$p = \frac{A_s}{bd}$ = steel ratio of longitudinal tension reinforcement
P = load on the beam
P_u = maximum load on beam
s = spacing of stirrups
$r = \frac{A_v}{bs}$ = ratio of web reinforcement

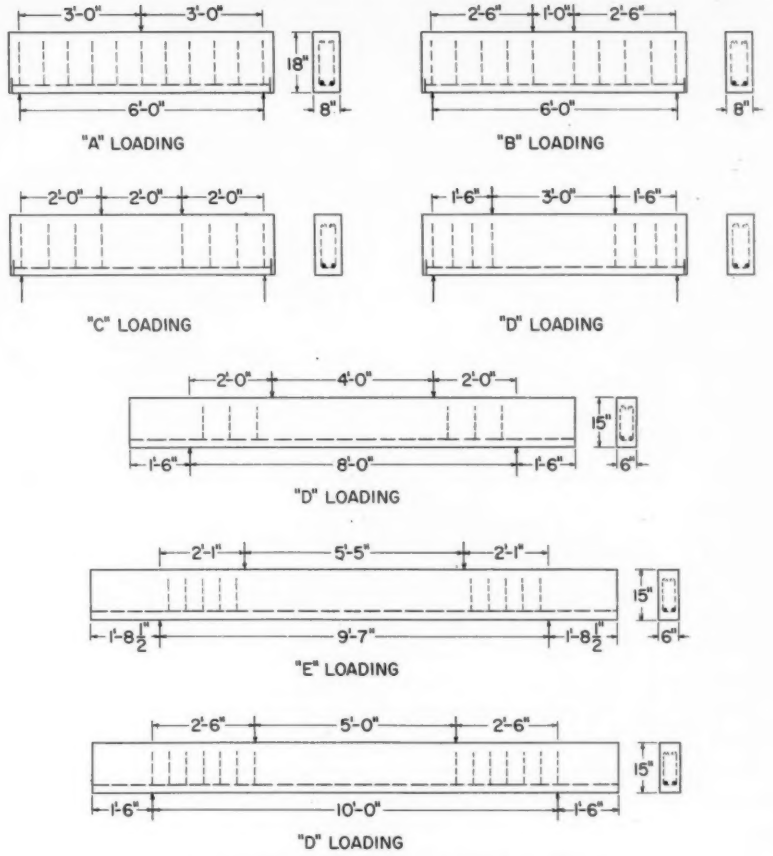

Fig. 3—Dimensions of beams and type of loading

$V = \frac{1}{2} P$ = end shear of beam

$V_u = \frac{1}{2} P_u$ = end shear at maximum load

$v = \dfrac{V}{bjd}$ = shearing stress at load P

$v_u = \dfrac{V_u}{bjd}$ = shearing stress at maximum load

$v_c = 7000\, p + (0.12 f_c')\dfrac{d}{a} + 2500\sqrt{r}$ = calculated shearing stress at maximum load

As a rule the beams with web reinforcement showed small cracks near the center of the span when the applied load was approximately 20 percent of the maximum load for the beam. These cracks were, in general, nearly

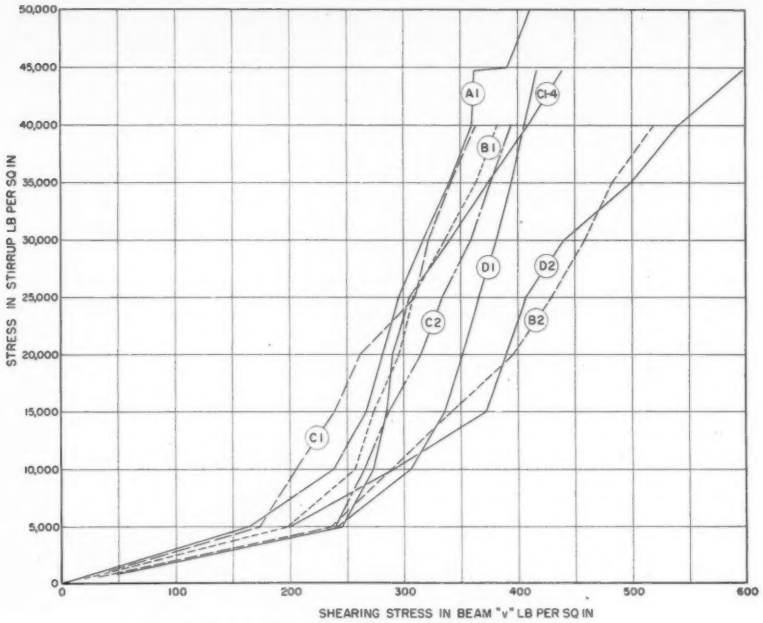

Fig. 4—Relation between shearing stress and stress in stirrups

vertical in the center portion of the beam and became inclined toward the load point between that point and the support. At a load of approximately 40 percent of the maximum the first crack which later developed into a diagonal tension crack appeared near the support. As more load was applied the diagonal crack gradually extended from a point near the support to a point near the load and finally opened to produce failure.

The beams with no web reinforcement showed initial cracks at loads a little less than those with web reinforcement and the first diagonal crack appeared at a considerably lower load. These beams failed suddenly by the opening of the diagonal crack.

A stress of 20,000 psi in a stirrup leg was reached at loads varying from 42 to 58 percent of the maximum for the beam depending on the type of loading and the ratio of web reinforcement. A stress of 40,000 psi was reached at loads ranging from 60 to 80 percent of maximum. Fig. 4 shows the relation between the shearing stress, v, in the beams for each type of loading and the average measured stress in the stirrups. After the yield point was reached in one stirrup leg the stress increased in adjacent stirrups indicating the tension was being taken up by the other stirrups. Fig. 5 shows a typical pattern of the stress variation in stirrups for increasing loads as measured by the SR-4 strain gages. The stresses plotted are the average for both legs of

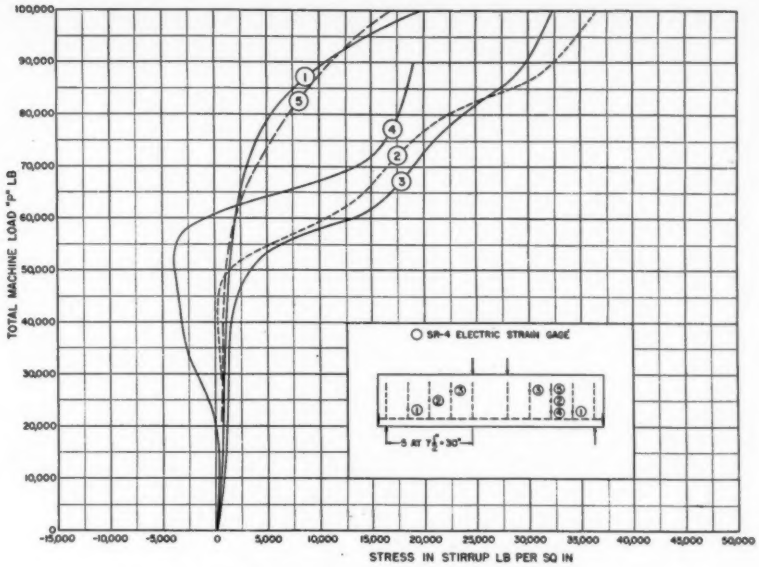

Fig. 5—Typical relationship between stirrups and applied load

the similarly located stirrups in both ends of the beam. Fig. 6 shows the deflection for beams under the different loadings. Fig. 7a and 7b are photographs of a few of the beams taken after failure which show the crack patterns.

Some previous investigations indicated that the manner of loading would affect the shear capacity of a beam. The results of this study, with tests under five different loading conditions, show definitely that the loading condition is an important factor.

More data than available at present are needed to evaluate some of the factors which determine the diagonal tension resistance of a beam. A study of the data from some previous tests by other investigators and the results of the beams tested made it possible to put together an expression which gives the maximum shearing stress for the beams in this program. This calculated shearing stress is given by the following empirical expression:

$$v_c = 7000\ p + (0.12\ f_c')\frac{d}{a} + 2500\ \sqrt{r}$$

This formula is *not for general application* but gives as well as might be expected the shear capacity of all beams tested which failed in diagonal tension. The calculated values, v_c, are in good agreement with the maximum shearing stress, v_u, determined from tests considering the well-known variability of concrete. The shearing stress developed in tests of duplicate beams differ in

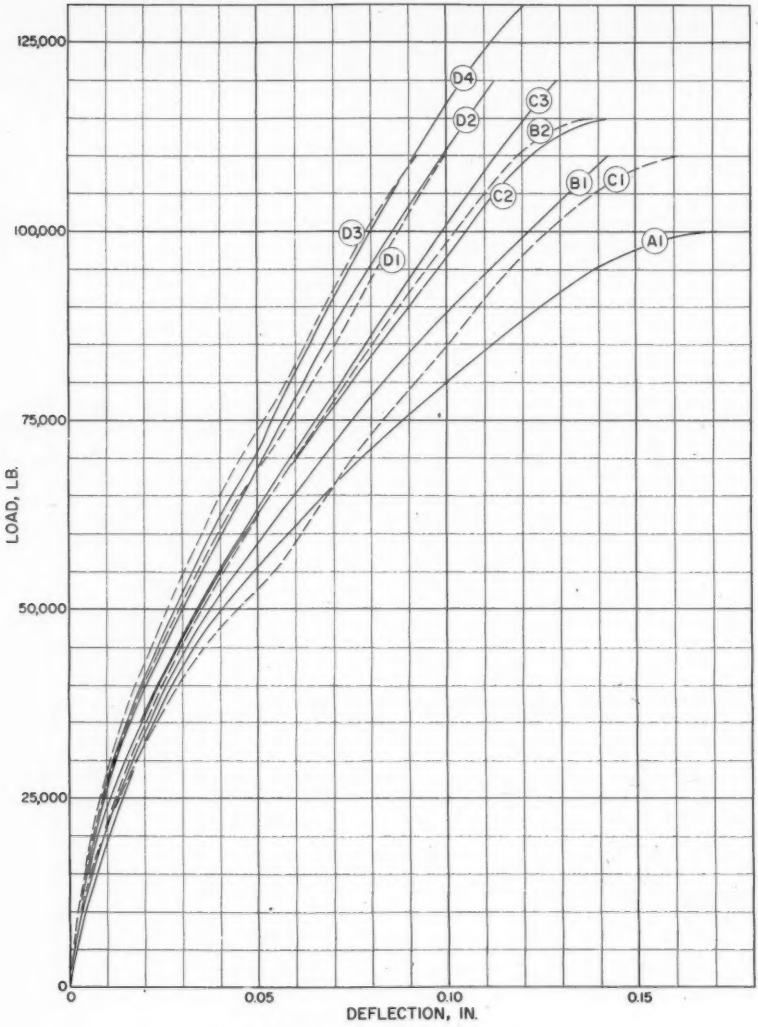

Fig. 6—Deflection of 8 x 18-in. beams

some cases more than the average difference between the values v_u and v_c for any group of similar beams. Table 2 gives the principal test results for individual beams and the relation between v_u and v_c. Included are only those beams which showed failure in diagonal tension but in some cases either apparent yielding of the tensile reinforcement or compression failure occurred

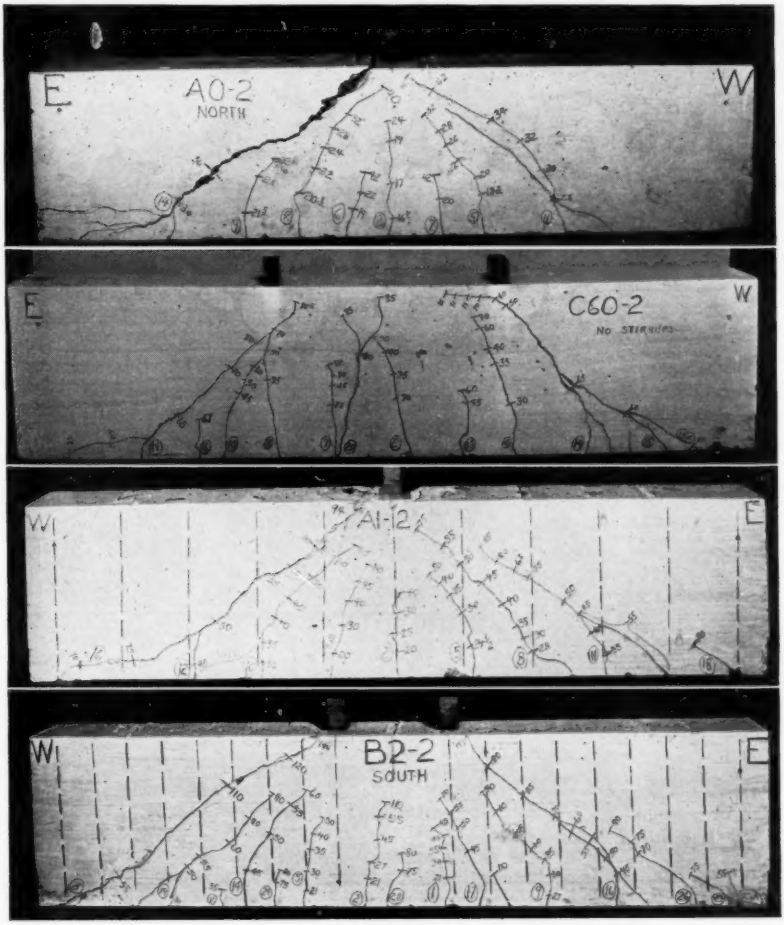

Fig. 7a—Typical beam cracks at failure

at the same time and in such cases the cause of failure is noted as tension or compression although it may be questionable as to the primary cause. In the designation of the beams the letter identifies the type of loading, the following numeral is used to differentiate between the ratios of web reinforcement and the last numeral is the specimen number. The average value of v_u for 62 beams is 574 psi and v_c is 575 psi which gives an average of $v_c/v_s = 1.002$. The tests include beams with two different cross sections, four different spans, five different loadings and concrete strengths from 2000 to 6900 psi.

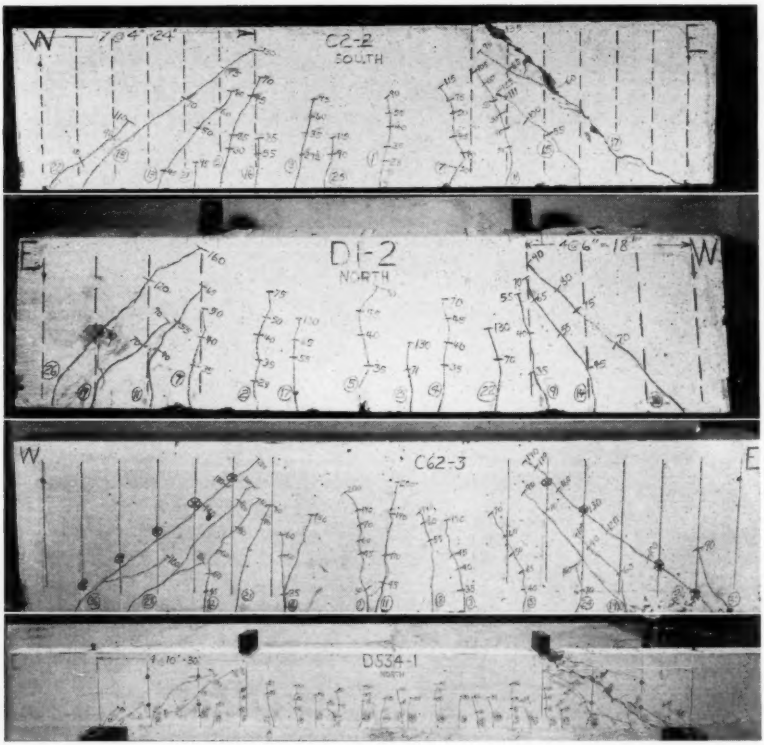

Fig. 7b—Typical beam cracks at failure

SUMMARY

Tests were conducted on beams designed to fail in diagonal tension. The specimens were of two different cross sections and four spans with varying ratios of web reinforcement, some variation in the concrete strength and percentages of tension reinforcement and tested under five different loading conditions. Resistance strain gages were used to determine the strains in the reinforcing bars, the deflections of beams were measured and cracks recorded for increasing loads up to failure.

The results are reasonably consistent for tests on concrete specimens and show good evidence that the shear capacity of a beam increases with the strength of the concrete when other factors are the same. For the same concrete strength the resistance to failure in diagonal tension increased as the loads were shifted from the center of the span toward the supports. More specifically, the strength in shear varied as the compressive strength multiplied by a factor representing the ratio of depth of beam to distance from the

plane of load to the plane of support. The resistance to shear was found to vary as the square root of the ratio of web reinforcement and the first power of the ratio of tensile reinforcement.

It was recognized that the 8 x 18-in. beam specimens having a 6-ft span had a high depth to span ratio. For this reason a limited number of specimens 6 x 15-in. beams having a 10-ft span were tested thus cutting this ratio in half and bringing it more nearly in line with the ratios used in practice. A study of the data resulted in a tentative formula which takes into account the variables mentioned and from which the shear capacity for both sizes of beams were calculated. The value of the maximum shearing stress in the beams of this series, calculated by the formula agree as well as can be expected with the values as determined by tests.

TABLE 2—RESULTS AND ANALYSIS OF BEAMS

Beam No.	f_c', psi	Tensile reinforcement		Stirrups		j	a, in.	P, lb	V_u, psi	V_c, psi	Ratio, v_c/v_u	Failure*
		Pieces & bar No.	p	s, in.	r							
8 ft x 18-in. Beams—Span 6 ft 0 in.												
A1-1	3575	3-No. 10	0.0310	7.2	0.0038	0.831	36	100,032	489	554	1.133	D.T.
2	3430	3-No. 10	0.0310	7.2	0.0038	0.830	36	94,032	461	547	1.187	D.T.
3	3395	3-No. 10	0.0310	7.2	0.0038	0.829	36	100,032	491	545	1.110	D.T.
4	3590	3-No. 10	0.0310	7.2	0.0038	0.830	36	110,032	538	555	1.032	D.T.
B1-1	3388	3-No. 10	0.0310	7.5	0.0037	0.829	30	125,370	615	577	0.940	D.T.
2	3680	3-No. 10	0.0310	7.5	0.0037	0.832	30	115,370	564	595	1.055	D.T.
3	3435	3-No. 10	0.0310	7.5	0.0037	0.829	30	128,061	628	580	0.924	D.T.
4	3380	3-No. 10	0.0310	7.5	0.0037	0.828	30	120,561	591	577	0.976	D.T.
5	3570	3-No. 10	0.0310	7.5	0.0037	0.831	30	108,561	531	589	1.109	D.T.
B2-1	3370	3-No. 10	0.0310	3.75	0.0073	0.828	30	135,370	665	638	0.959	D.T.
2	3820	3-No. 10	0.0310	3.75	0.0073	0.835	30	144,870	705	665	0.943	D.T.
3	3615	3-No. 10	0.0310	3.75	0.0073	0.832	30	150,561	736	653	0.887	D.T.
B6-1	6110	3-No. 10	0.0310	7.5	0.0037	0.860	30	170,561	806	745	0.924	D.T.
C1-1	3720	2-No. 10	0.0207	8.0	0.0034	0.855	24	124,870	594	577	0.971	D.T.
2	3820	2-No. 10	0.0207	8.0	0.0034	0.856	24	139,870	664	584	0.880	D.T.
3	3475	2-No. 10	0.0207	8.0	0.0034	0.851	24	110,561	528	558	1.057	D.T.
4	4210	2-No. 10	0.0207	8.0	0.0034	0.862	24	128,561	606	614	0.982	D.T.
C2-1	3430	2-No. 10	0.0207	4.0	0.0069	0.851	24	130,370	623	616	0.989	D.T.
2	3625	2-No. 10	0.0207	4.0	0.0069	0.854	24	135,370	644	631	0.980	D.T.
3	3500	2-No. 10	0.0207	4.0	0.0069	0.852	24	145,561	695	622	0.895	T
4	3910	2-No. 10	0.0207	4.0	0.0069	0.858	24	129,561	614	653	1.064	D.T.
C3-1	2040	2-No. 10	0.0207	8.0	0.0034	0.822	24	100,561	497	447	0.899	D.T.
2	2000	2-No. 10	0.0207	8.0	0.0034	0.820	24	90,061	446	444	0.996	D.T.
3	2020	2-No. 10	0.0207	8.0	0.0034	0.822	24	84,561	418	446	1.067	D.T.
C4-1	3550	3-No. 10	0.0310	8.0	0.0034	0.830	24	139,061	681	636	0.934	D.T.
C6-2	6560	3-No. 10	0.0310	8.0	0.0034	0.863	24	190,561	898	867	0.965	D.T.
3	6480	3-No. 10	0.0310	8.0	0.0034	0.863	24	195,561	921	861	0.935	D.T.
4	6900	3-No. 10	0.0310	8.0	0.0034	0.867	24	192,717	903	894	0.990	D.T.
D1-1	3800	2-No. 9	0.0163	6.0	0.0046	0.868	18	135,370	634	673	1.062	D.T.
2	3790	2-No. 9	0.0163	6.0	0.0046	0.868	18	160,370	751	672	0.895	T
3	3560	2-No. 9	0.0163	6.0	0.0046	0.865	18	115,370	542	649	1.197	D.T.
D2-1	3480	2-No. 9	0.0163	4.5	0.0061	0.864	18	130,370	613	666	1.086	D.T.
2	3755	2-No. 9	0.0163	4.5	0.0061	0.868	18	140,370	657	694	1.056	D.T.
3	3595	2-No. 9	0.0163	4.5	0.0061	0.866	18	150,370	705	678	0.960	T
4	3550	2-No. 9	0.0163	4.5	0.0061	0.865	18	150,561	708	673	0.951	T
D3-1	4090	3-No. 9	0.0244	3.0	0.0092	0.852	18	177,561	847	830	0.980	D.T.
D4-1	3350	2-No. 9	0.0163	2.25	0.0122	0.862	18	140,370	662	734	1.109	D.T.

*Failure marked "D.T." indicates the beam failed in pure diagonal tension. In a few cases marked "T" there appeared to be yielding at the tensile reinforcement at the time of diagonal tension failure.

Continued on p. 156

TABLE 2 (Cont'd)—RESULTS AND ANALYSIS OF BEAMS

Beam No.	f'_c, psi	Tensile reinforcement		Stirrups		j	a, in.	P, lb	V_u, psi	V_c, psi	Ratio, v_c/v_u	Failure*
		Pieces & bar No.	p	s, in.	r							
6 x 15 in. Beams—Span 8 ft 0 in.												
D1-6	4010	2-No. 10	.0342	8.0	.0046	0.832	24	78,561	636	657	1.033	D.T.
7	4060	2-No. 10	.0342	8.0	.0046	0.833	24	80,561	651	660	1.014	D.T.
8	4030	2-No. 10	.0342	8.0	.0046	0.833	24	83,561	676	658	0.973	D.T.
6 x 15-in. Beam—Span 9 ft 7 in.												
E1-2	4375	2-No. 10	.0342	5.0	.0073	0.837	25	99,717	802	713	0.889	D.T.
6 x 15 in. Beams—Span 10 ft 0 in.												
D2-6	4280	2-No. 10	0.0342	6.0	0.0061	0.836	30	75,717	610	646	1.059	D.T.
7	4120	2-No. 10	0.0342	6.0	0.0061	0.833	30	70,717	572	639	1.117	D.T.
8	3790	2-No. 10	0.0342	6.0	0.0061	0.829	30	75,717	615	622	1.011	D.T.
D4-1	3970	2-No. 10	0.0342	7.5	0.0049	0.831	30	75,717	614	611	0.995	D.T.
2	3720	2-No. 10	0.0342	7.5	0.0049	0.828	30	70,717	575	599	1.042	D.T.
3	3200	2-No. 10	0.0342	7.5	0.0049	0.819	30	74,217	610	573	0.939	D.T.
D5-1	4020	2-No. 10	0.0342	10.0	0.0037	0.832	30	65,717	532	590	1.109	D.T.
2	4210	2-No. 10	0.0342	10.0	0.0037	0.835	30	70,717	570	600	1.053	D.T.
3	3930	2-No. 10	0.0342	10.0	0.0037	0.831	30	70,717	573	586	1.023	D.T.
8 x 18-in. Beams with no web reinforcement—Span 6 ft 0 in.												
A0-1	3120	2-No. 7	0.0098	..	...	0.884	36	40,032	184	229	1.245	D.T.
2	3770	2-No. 7	0.0098	..	...	0.892	36	48,532	221	262	1.186	D.T.
3	3435	2-No. 7	0.0098	..	...	0.888	36	53,532	245	246	1.004	T
B0-1	3420	2-No. 7	0.0098	..	...	0.887	30	54,420	249	279	1.120	D.T.
2	3468	2-No. 7	0.0098	..	...	0.888	30	42,370	194	282	1.454	D.T.
3	3410	2-No. 7	0.0098	..	...	0.887	30	57,561	264	278	1.053	D.T.
C0-1	3580	2-No. 7	0.0098	..	...	0.889	24	78,370	358	344	0.961	D.T.
2	3405	2-No. 7	0.0098	..	...	0.887	24	79,870	366	330	0.902	T
3	3420	2-No. 7	0.0098	..	...	0.887	24	75,061	344	332	0.965	D.T.
D0-1	3750	2-No. 7	0.0098	..	...	0.892	18	99,620	454	453	0.998	D.T.
2	3800	2-No. 7	0.0098	..	...	0.892	18	116,870	533	458	0.859	T
3	3765	2-No. 7	0.0098	..	...	0.892	18	100,370	457	455	0.996	D.T.

*Failure marked "D.T." indicates the beam failed in pure diagonal tension. In a few cases marked "T" there appeared to be yielding of the tensile reinforcement at the time of diagonal tension failure.

Discussion of a paper by Arthur P. Clark:

Diagonal Tension in Reinforced Concrete Beams*

By PHIL M. FERGUSON, ORESTE MORETTO, I. E. MORRIS, DOUGLAS E. PARSONS, GROVER L. ROGERS, ROBERT S. ROWE and AUTHOR

By PHIL M. FERGUSON†

This discussion relates entirely to beams without web reinforcement, namely, the twelve 8 x 18-in. beams reported at the end of Table 2. The author notes that the manner of loading is definitely a factor in shear capacity of beams and even includes the position of load as one of the factors in his empirical relationship for shear strength. The writer believes this matter needs even greater emphasis.

Professors Turneaure and Maurer in 1909 reported tests at the University of Wisconsin‡ on miniature mortar beams with 1.41 percent of reinforcement tested with symmetrical loads at various distances from supports. The shearing strengths varied from 205 psi for a center load to 1200 psi for loads just 2 in. from the supports. Although they suggest that this lowered shear strength for loads near the center of span shows the effect of stress in the horizontal steel, calculation shows that the bending moment is nearly as large when capacity loads are placed near the support. The increased values for loads near supports would not seem to be due entirely to small steel stress.

In general these results confirm those of Mr. Clark on full sized beams. A conventional calculation of steel stress for his beams when loads are 18 and 24 in. from supports shows stresses in excess of the yield point. Failures due to yielding of steel are noted in three out of the four series in this group. Hence low steel stress cannot be the reason for the extra strength with loads near the supports. Nor can the variation in resistance be explained by dowel action of the steel or by percentage of steel; steel was the same in all tests.

The varying factor must be related to shear distribution in the vicinity of reactions and concentrated loads, as well as some pronounced vertical compressive forces. Footing tests have clearly shown that shear at the face of the column is not the shear which determines diagonal tension failure. When stirrups are not used in beams, possibly engineers need to recognize that the design shear for diagonal tension is not the shear at the face of support, but the shear at some distance out from the support. It is not believed that the critical section for uniform loads will be as far from supports as these tests with concentrated loads might seem to indicate. However, the distance to the

*ACI JOURNAL, Oct. 1951, *Proc.* V. 48, p. 145. Disc. 48-11 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 24, No. 4, Dec. 1952, Part 2, *Proceedings* V. 48.

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‡Turneaure, F. E., and Maurer, E. R., *Reinforced Concrete Construction*, 2nd Edition, 1909, p. 166; 3rd Edition, 1919, p. 145.

critical section might well be as much as d or $1.5d$ from the support. Timber designers have for a number of years recognized a similar condition in connection with the location of concentrated loads for maximum horizontal shear near supports. Although this is commonly ascribed to other factors, the writer believes that vertical compressive stresses near the reactions, when combined with a totally different shear distribution, constitute the chief reason for increased shear resistance of timber beams as well as for concrete footings and concrete beams. This matter needs further study, but it might well lead to a change in design specifications.

The writer is less concerned with this increased shear capacity near supports than he is with reduced shear capacity away from supports. To visualize the results of these tests, Fig. A has been plotted from the average of Mr. Clark's test results. The point on the extreme left is possibly 10 percent too high for an exact comparison because the concrete for this series was of higher quality; but this is of minor consequence. The curve does definitely emphasize the increased strength near supports. It likewise emphasizes the fact that the minimum strength may well be lower for loads farther from the support than those reported here. The tests at the University of Wisconsin showed strengths for a mid-span load only some 60 percent of that when the load was about $2d$ from the supports. Professor Talbot in 1909 also reported* that shear strength of beams without stirrups depended upon the ratio of length to depth. Since all beams were the same depth and were loaded at third points, this corresponds to loads at distances from the support which varied from $2.4d$ to $4.8d$. He shows curves based on averages from his 1908 tests for several percentages of steel. These all show shear strengths for loads $4.8d$ from support 80 to 84 percent as large as those for loads $2.4d$ from support.

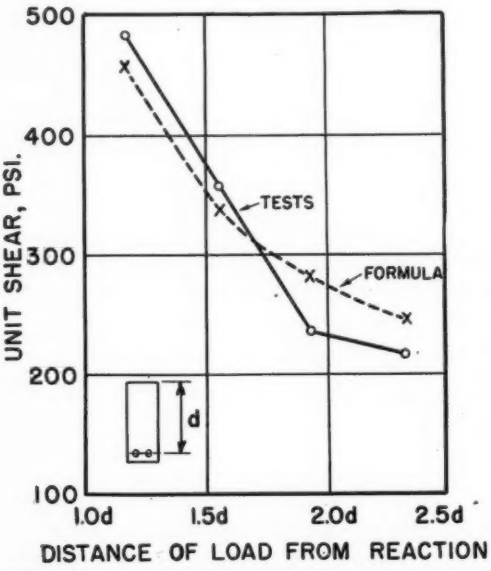

Fig. A—Variation in shear strength with position of load, no stirrups

It appears quite doubtful that present specifications maintain an adequate factor of safety in the allowable shear value of $v = 0.03f_c'$, especially when

*Talbot, A. N., "Tests of Reinforced Concrete Beams; Resistance to Web Stresses. Series of 1907 and 1908," *Bulletin* No. 29, University of Illinois Engineering Experiment Station, Jan. 1909, p. 45.

concentrated loads are involved. Possibly this allowable was based in part on tests where loads were too close to the support. It is noted that Mr. Clark reported an average ultimate shear of only $0.0630f_c'$ (minimum specimen $0.0535f_c'$) when the load was 36 in. or $2.3d$ from support. In the light of the other tests reported above, it appears that loads still farther from the support would probably lead to values only some 80 percent this large, that is, possibly $0.050f_c'$ average ($0.043f_c'$ minimum). The specification allowable of $0.03f_c'$ is too close to these values for comfort. This is also a matter of some concern in beams with stirrups, because stirrups are often discontinued where v drops to $0.03f_c'$, which is usually a considerable distance from the support.

It should be noted that the author's empirical formula was designed for stirrups and does not fit these beams without stirrups too well. Fig. A would seem to indicate that it should probably not be used for these beams without stirrups, especially for loads $2d$ or farther from supports.

This discussion is in no sense a criticism of Mr. Clark's excellent work and paper; he has brought out some important facts. The writer believes they are important enough to require a re-examination of our design specifications and probably some supplementary tests. He would not like to see these test results filed away without such action.

By ORESTE MORETTO*

The tests reported by Mr. Clark are a timely contribution to a reinforced concrete problem that was never solved adequately. To the writer's knowledge, this is the third series of tests† in which beams intended for diagonal tension research failed by a definite diagonal tension failure. Due to a misconception of the behavior of reinforced concrete beams, most of the other specimens tested for this purpose were designed in such a way that initial failure by tension or bond played an important part in their behavior, resulting in a failure due to the simultaneous effect of tension, bond and diagonal tension. In the tests under discussion, bond failure was prevented by anchors at the end of tension bars. Tension failure was also avoided, in most of the specimens, by using a high percentage of tension steel.

Results obtained from these tests confirm a fundamental misconception existing in most reinforced concrete building regulations all over the world. These regulations impose a maximum allowable shearing stress, regardless of the amount of web reinforcement provided in the beam. Danger of excessive detrimental cracking is the reason advanced for this limitation. It is, consequently, based on the concrete tension resistance.

To the writer's knowledge, the only exception to this rule appears to be the regulations of the Ministère de la Reconstruction et de l'Urbanisme of France.‡ The maximum shearing stress specified is equal to one half the

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†The other two were reported by: W. A. Slater, A. R. Lord and R. R. Zipprodt, "Shear Tests of Reinforced Concrete Beams," Technical Paper No. 314 of the Bureau of Standards; and by Oreste Moretto, "An Investigation of the Strength of Welded Stirrups in Reinforced Concrete Beams," ACI JOURNAL, Nov. 1945, *Proc.* V. 42, p. 141.

‡"Règles d'Utilization du Béton Armé Applicable aux Travaux Dépendant du Ministère de la Reconstruction et de l'Urbanisme et aux Travaux Privés," Mar. 1948.

allowable compression stress (expressed in terms of the ACI standard this value would amount to about $v = 0.16f_c'$). It is based on other factors than the concrete tension resistance, possibly on a study of the *rupture line* of concrete.

Existing diagonal tension tests seem to indicate beyond doubt that the maximum shearing stress inserted in most reinforced concrete regulations is not justified. Within the practical limits of a well designed beam of normal dimensions, concrete will never fail by shear; failure will always occur by diagonal tension. Since it is always possible to place sufficient web reinforcement to avoid this kind of failure without the appearance of detrimental cracks, there is no need for a limit to the maximum allowable shearing stress. The situation might be different in very short beams. Failure might take place by shearing off the concrete or by diagonal compression in heavily reinforced beams of this type. For this reason, one or both of these phenomena should serve as a criterion for determining the maximum allowable shearing stress. The tests by Slater, Lord and Zipprodt might give some idea of the value of this maximum. Unfortunately, Technical Paper No. 314 is not available to the writer and he cannot comment on this matter further. In any event, more test results would be needed for this purpose.

The writer does not understand clearly how Mr. Clark arrived at the formula that expresses the calculated stress v_c that interprets the results of tests at maximum load. Above all, it surprises him that, apparently, no attempt was made to derive a formula for general application. It would have given the tests a much greater value as a tool for general use.

With the above purpose in mind, the results of tests have been re-analyzed in relation to some of the variables studied. In Fig. B, beams with constant ratio of web reinforcement were selected. The value of $v_u - rf_v$, f_v being the yield point of the stirrup bars, was plotted against steel ratio of longitudinal tension reinforcement. Through the points corresponding to beams with equal web reinforcements, lines have been passed to show the general trend indicated by the tests. Fig. B shows that there is too great a scattering to draw any conclusion from it, but if it does indicate anything, it would seem to show that

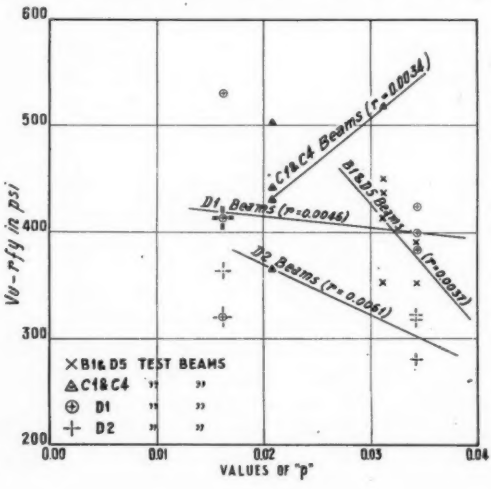

Fig. B—Values of $v_u - rf_v$ as a function of the ratio of tension steel for beams with equal web reinforcement

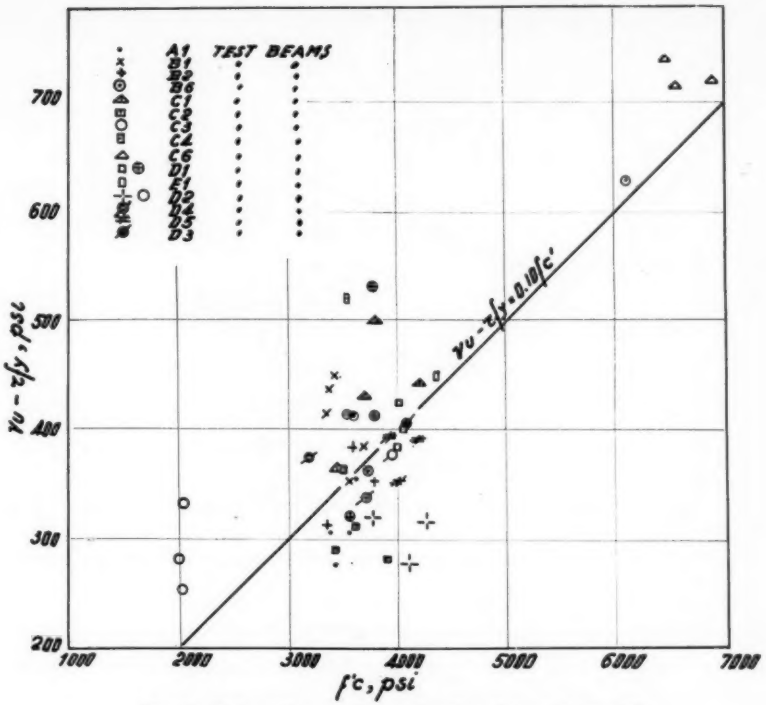

Fig. C—Values of $v_u - rf_y$ as a function of concrete strength

no relation can be found between shearing stress and ratio of main reinforcement.

In Fig. C the values of $v_u - rf_y$ have been plotted against concrete strength, f'_c. Although there is much scattering, a straight line has been passed through the points with the view of deriving a formula for general practical application. The formula is:

$$v_{uc} = rf_y + 0.10f_c' \quad \ldots\ldots (1)$$

Eq. 1 might not agree with the results of tests as well as that of the author (maximum difference between actual tests values and calculated values equals 26 percent) but it is simple enough to be of general use and it interprets the tests with sufficient accuracy for practical purposes (scattering of tests supposed to be identical is higher than the difference resulting between Eq. 1 and actual test results). The author's formula is too cumbersome.

Fig. D shows the line representing Eq. 1 against the points that indicate actual test values as a function of the ratio of web reinforcement, which confirms what has been said about it.

In the analysis made in Fig. B, C and D, tests D4 and those of beams with no web reinforcement have been omitted. Each beam type has been

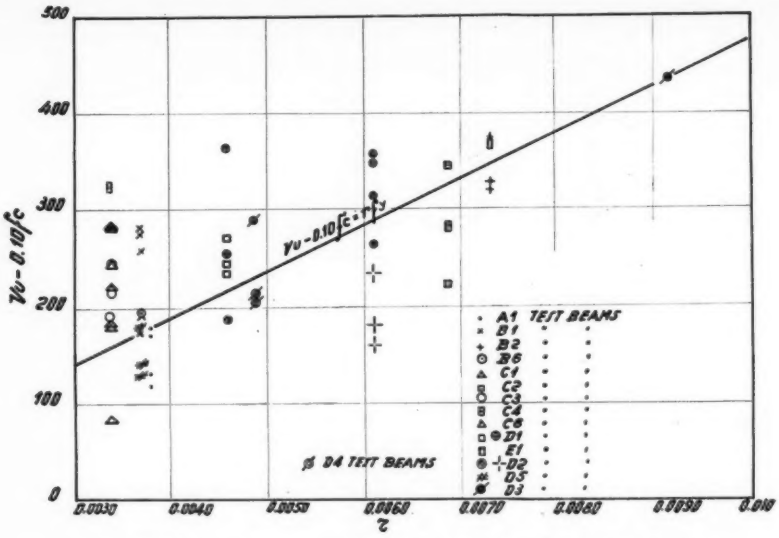

Fig. D—Values of $v_u - rf_y$ as a function of the ratio of web reinforcement

represented with a different sign, to show the dispersion of tests values from supposedly identical or nearly identical specimens.

By I. E. MORRIS*

Mr. Clark has given some valuable information on the subject of diagonal tension and effectiveness of vertical stirrups.

While the action of vertical stirrups in reinforced concrete beams agrees with test data about as well as could be expected, it is doubtful that this arrangement of shear reinforcement is the best we can devise.

Fig. E (a) shows a short section of beam; Fig. E (b) shows the distribution of shearing stresses as represented in textbooks. Fig. E (c) shows a stress distribution which, because of its simplicity, was chosen for this discussion. In both, V and V/jd are the same. It is obvious that in (c) $V/jd \times jd$ must equal V, which makes the horizontal shear equal to the vertical shear. These two forces acting together produce diagonal tension. This tension causes the concrete between the tension steel and the center of gravity of the compressive forces to function somewhat as the web members of a parallel-chord Pratt truss. The effect on the concrete is clearly shown in Fig. F, which shows diagonal tension cracks approximately as represented in test specimen B 2-2, Fig. 7a, of Mr. Clark's paper. In Fig. F, tension steel, and shear reinforcement as suggested, have been added to the illustration.

Note that the first two vertical stirrups at each end of the test beam have contributed little toward the prevention of diagonal tension cracks, while

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Fig. E—Stress distribution in beam

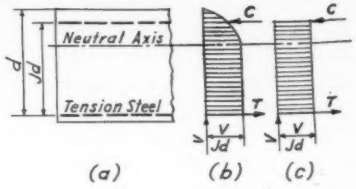

Fig. F—Method of reinforcing for diagonal tension

all of the curved bars are in position to function effectively over the full length of the beam. If the beam length were doubled and a uniform load substituted, the curved bars would not necessarily be extended over the full span. They could be placed at each end approximately as shown and anchored beyond the point where the concrete alone would absorb all diagonal tension.

It may be argued that the proposed method of reinforcing for shear would require more steel, and that placing of the bars would be more difficult. However, if we wish to reinforce effectively against diagonal tension, it will be necessary to provide whatever amount of steel is required. On the average job the difference in tonnage would be negligible and the fabrication would be simplified. In fact, if the stirrups in the test beam are #3 bars, their weight is nearly the same as the weight of ten #3 curved bars.

That the placing of bars would be more difficult is debatable. Such bars would normally be #2 or #3. They could easily be draped to form approximately a pattern indicated on the contract drawings, with #2 vertical stirrups about 3 ft on centers to hold them in position. An exact curvature is not essential. A given number of bars distributed over the distance *jd*, and curving slightly downward, is all that is required. At an interior support, the placing would be easier, since the bars could be supported at their midpoint and allowed to bend downward under their own weight.

If all tensile reinforcement in a simple beam under uniform load were placed so as to form a true parabola, there would theoretically be no shear in the concrete. It is this condition which we are approximating; but, because of the ability of concrete to resist some tension, we do not need a high degree of accuracy in placing the curved bars. The use of such bars, however, is recommended only for members having rather high shearing stresses. Obviously, if, according to our present procedure, only two or three vertical stirrups were required at each end of a beam, there would be no point in trying to use another type of shear reinforcement.

For the calculation of the required area of the curved bars, almost any amount of refinement could be used since each one of them is a different depth below the center of gravity of the compressive areas. But, referring

again to Fig. E (c), if we say that the total horizontal shear is equal to the total vertical shear, and if we neglect tension in the concrete and say that the horizontal shear exerts a direct pull on the bars, the area of these bars would be V/f_s. Therefore, if V = 20,000 lb and f_s = 20,000 psi, we would have 1 sq in. as the required area of the curved bars. This could be provided by five #3 bars in each face of the beam.

It is recognized that the foregoing is not an exact analysis, for each of the curved bars acts with a varying lever arm, and the bars are not parallel to the horizontal axis; also, the tensile value of concrete could be included, but 1 sq in. of diagonal reinforcement seems reasonable for the assumed V.

By DOUGLAS E. PARSONS*

The many published papers discussing diagonal tension in reinforced concrete beams is evidence of the widespread interest in the subject. Probably many investigators have tried to derive theoretical and empirical relations which could be used to make quantitative estimates of the effects of some of the factors on the shearing resistance of reinforced concrete beams. And it seems probable that others, like myself, reluctantly were forced to abandon their attempt to derive improved rules or equations for estimating the resistance of beams to diagonal tension failures. One of the chief obstacles to obtaining quantitative data on the effects of each of the important factors has been the inability to adjust original data for the effect of differences in the ratio of the depth of the beam to the span. No doubt, all who have seriously investigated the subject have sensed that the bending stresses affect the magnitude of the computed shearing stresses at failure but none heretofore has had the means for estimating the magnitude of the effect. The data in this paper may provide a method that will enable future research workers to re-analyze and to correlate much of the data in the literature.

A study of available data conducted several years ago indicated that the shearing strength of beams without web reinforcement was not a linear function of the compressive strength of the concrete; this fact had been reported by a number of investigators. Others had called attention to the non-linear relation between compressive and tensile strength of concrete, and various authors had suggested an equation of the form, $T = a(f_c')^b$, in which T = tensile strength of concrete, f_c' = compressive strength of concrete, a = a constant, and b = a constant whose value is greater than 0.5 and less than 1.0. Perhaps the following equation fits the available data about as well as any other equally simple one: $T = 1.5\ (f_c')^{3/4}$.

It would be natural to expect that a similar relationship exists between compressive strength of concrete and computed shearing stress at failure by diagonal tension of beams and that such an equation might be applicable to a wider range of strengths of concrete than the linear relation commonly assumed.

Data in the literature were too limited to afford reliable evidence on the relations that may exist between shearing resistance of beams and strength

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of concrete. However, neither theory nor available test data seemed to warrant equations of the forms suggested by Moretto and Clark, despite their present usefulness. For example, the diagonal tension resistance of beams without web reinforcement seemed to increase with an increase of p at a rate somewhat less than that implied by these linear equations.

Inasmuch as the data presented in the paper will be useful to others who study the factors affecting the diagonal tension resistance of reinforced concrete beams, it is important that all of the information which is likely to be so used be made available. It seems probable that investigators would appreciate the inclusion in the author's "closure" of a compact table of averages of the data in Table 2 for like beams. This table should include the average effective depth of the beams. Values for v_c and v_c/v_u could be omitted to allow space for values of d and d/a.

By GROVER L. ROGERS*

The author has presented experimental data which throws much light on the problem of explaining such diverse effects as concrete strength, percentage of longitudinal reinforcing, percentage of web reinforcing, and the location of concentrated loads on the ultimate shear capacity of reinforced concrete beams.

Until now there has probably been considerable doubt in the minds of some engineers regarding the first two of these. Such doubts should be diminished after careful study of the author's data. The third effect, percentage of web reinforcing, has always been considered as being the important factor in the design for diagonal tension. On this point, the author's data will serve as a possible check on evaluating the correctness of currently employed methods of analyzing this factor. The fourth effect, location of concentrated loads, may be studied advantageously from the point of view of pure mechanics and contrary to the author's statement in the synopsis, it has received attention in the technical press. A notable example of such a discussion is that given by Prof. G. Magnel in his four volume treatise on the theory and design of reinforced concrete structures.† Magnel considers the problem of sufficient importance to warrant an entire chapter on its discussion under the heading of "extraordinary beams," which by his definition are those having large ratios of depth to length or those which are subjected to heavy concentrated loads in the region of a support. In either case, the theory is the same and the critical diagonal tension is shown to be much less than that normally assumed since the vertical normal stress on an element at the neutral axis is of appreciable magnitude, due to the localized effect of the loading, and consequently must be combined with the shearing stress acting on the element. Construction of a simple Mohr's circle is sufficient to illustrate the point.

The following example is based on Magnel's discussion and shows how

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†Magnel, G., *Pratique du Calcul du Beton Arme*, Editions Fecheyr, Ghent, Belgium, 5th Edition, V. 1, 1949, pp. 146-171.

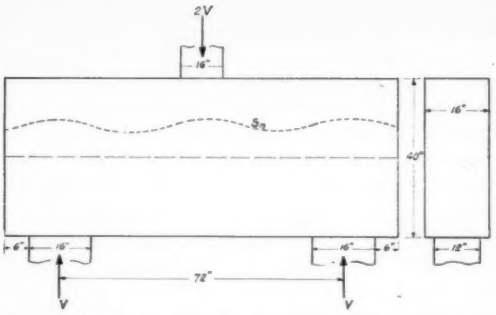

Fig. G—Distribution of normal stresses on the neutral axis of a beam

such problems may be treated. Consider the beam shown in Fig. G; the dotted line represents a plausible distribution of normal stresses on the neutral axis of the beam. The true distribution is difficult and probably impossible to find, but in this and in most cases a reasonable distribution may be assumed with some confidence. The average ordinate of the curve shown in Fig. G will be

$$s_n \text{ (average)} = \frac{2\,V}{100 \times 16} = 0.00125\,V \text{ psi}$$

It is known that diagonal tension will be the most critical when this stress is the least. An estimate of the minimum pressure from the curve may be taken as 0.8 times the average, or

$$s_n \text{ (minimum)} = 0.8 \times 0.00125\,V = 0.001\,V \text{ psi}$$

Since the shear on any section is constant and equal to V, one finds that the maximum vertical shearing stress is

$$v_{max} = \frac{V}{bjd} = \frac{V}{16 \times 0.875 \times 38} = 0.00188\,V \text{ psi}$$

using $j = 0.875$.

If a Mohr's circle is now drawn using the above for the normal and shearing stresses (Fig. H), one finds that the critical tension is 0.00145 V psi. In relation to the value of 0.00188 V, this represents a 23 percent reduction or a load capacity which is 30 percent higher than that found assuming that diagonal tension is equal to vertical shearing stress.

Also, the author gives an empirical formula for determining an equivalent shearing stress which would be present in a given beam at failure. This stress is to be used with the conventional formula $v_{max} = V/bjd$ for predicting shear capacity of the beam. He wisely points out that such an expression, based purely on the observed data can not be considered suitable for general use. It would seem desirable to have, therefore, a semi-empirical expression of his general type which would have as its foundation a rational analytical development and which would include in its constitution certain constants which would depend on a-

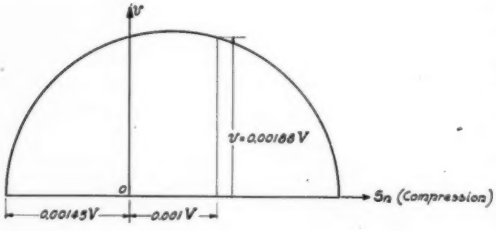

Fig. H—Mohr's circle for normal and shearing stresses

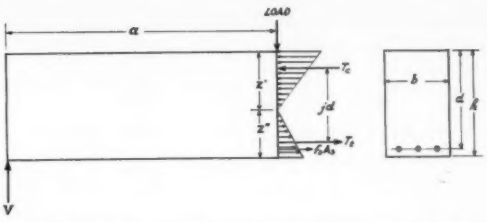

Fig. I—Reinforced concrete beam reinforced with only longitudinal steel

vailable experimental data.

Fig. I represents a reinforced concrete beam reinforced with longitudinal steel only. If concrete is assumed to be capable of working in tension as well as in compression, then the pressure diagrams on any cross section will be as those shown. The author's notation has been used whenever possible. Exceptions or additions are as follows:

E_c and E_s = moduli of elasticity of concrete and steel, respectively. The modulus of elasticity of concrete in tension is taken as equal to its modulus in compression.

z' and z'' = distance from neutral axis to extreme top and bottom fibers, respectively.

f_c, f_t, f_s = concrete compressive stress on top fiber, concrete tensile stress on bottom fiber and steel stress, respectively.

$n = E_s/E_c$; $k = z'/h$

T_t = total tensile area of pressure diagram due to contribution from steel as well as concrete tensile stresses.

Considering the relations between stress in the steel and the compressive stress in concrete and also the relation between tensile stress in the concrete and compressive stress in the concrete, it can be shown that

$$f_s = n f_c \left(\frac{d}{z'} - 1\right)$$
$$f_t = f_c \left(\frac{z''}{z'}\right) \qquad \text{.....(1)}$$

And if the pressure diagram for compression is equated to the pressure diagram for tension, one finds that

$$\frac{b f_c z'}{2} = \frac{b f_t z''}{2} + A_s f_s \qquad \text{.....(2)}$$

$$\text{or } k = p n \left(\frac{d}{h}\right) - p n k + \tfrac{1}{2} \qquad \text{.....(2a)}$$

where p is the percentage of longitudinal steel. If the value of d is taken as $0.9\ h$, Eq. 2a then becomes

$$k = \frac{1.8\ pn + 1}{2\ (1 + pn)} \qquad \text{.....(3)}$$

Referring once again to Fig. I, it may be seen that the total tensile force acting on the section is

$$T_t = \frac{b f_t z''}{2} + A_s f_s \qquad \text{.....(4)}$$

and when $d = 0.9\ h$, it becomes

$$T_t = b f_t h \frac{1 - 2k + k^2 + 1.8\ pn - 2\ pnk}{2\ (1 - k)} \qquad \text{.....(4a)}$$

Also from Fig. I it may be seen that the moment M at the section is equal

to Va and also equal to $T_t\,jd$. This means that for the shearing stress at any point on the neutral axis between the support and the load, one has

$$v_{max} = \frac{V}{bjd} = \frac{M}{abjd} = \frac{T_t}{ab} \quad \ldots\ldots (5)$$

Substituting Eq. 4a into Eq. 5, there follows

$$v_{max} = 0.25\, f_t \left(\frac{h}{a}\right) \frac{1 + 3.6\, pn + 3.24\, (pn)^2}{1 + 1.2\, pn + 0.2\ \ (pn)^2} \quad \ldots\ldots (6)$$

If the term in brackets in Eq. 6 is plotted against values of the product pn, it may be seen that a simpler expression which approximates it closely is $(1 + 2.5\, pn)$. And substituting this into Eq. 6, one finds

$$v_{max} = 0.25\, f_t \left(\frac{h}{a}\right) (1 + 2.5\, pn) \quad \ldots\ldots (7)$$

This is an expression for maximum shearing stress at any point on the neutral axis throughout the distance a in terms of tensile stress in the concrete at the load point. It is not necessarily equal to the value of the diagonal tension, for reasons explained earlier, and like all formulas based on the theory of elasticity it is valid only as long as no cracks are present in the beam.

It is generally assumed that the strength of concrete in tension is equal to $\frac{1}{10}$ its value in compression. Introducing the results of such an assumption into Eq. 7 and letting v'_{max} represent the shearing stress at the load causing the first tensile crack to form, there follows

$$v'_{max} = 0.025\, f_c' \left(\frac{h}{a}\right) (1 + 2.5\, pn) \quad \ldots\ldots (8)$$

Let it be assumed that complete rupture of such a beam will occur when there exists an equivalent shearing stress which is some multiple B times the stress represented by Eq. 8. The factor B must be found experimentally. Using the well established rule-of-thumb that the modulus of elasticity of concrete is equal to 1000 times its compressive strength and also the fact that a good value for the modulus of steel is 30,000,000 psi, one finds that the equivalent shearing stress at rupture, v''_{max}, is

$$v''_{max} = B \left(\frac{h}{a}\right) (0.025\, f_c' + 1875\, p) \quad \ldots\ldots (9)$$

If web reinforcing is present in the beam, it becomes necessary to expand Eq. 9 to account for the effect of that reinforcing on the shearing stress. The term to be added may be found by considering that as long as the load is at a distance a equal to or greater than $1.5\, h$ then a possible rupture plane will be inclined at an angle of 45 degrees. In such a case, the number of stirrups crossing the plane will be equal to the horizontal projection of the plane divided by the spacing of the stirrups s. Consider that the horizontal projection will be equal to jd which recognizes that any crack will tend to become vertical near the top and bottom of the beam due to the lack of web resistance in those areas. The total shear on the section will then be

$$V = \frac{A_w\, f_s\, jd}{s}$$

$$\text{and } v_{max} = \frac{A_w f_s}{sb} = r f_s \quad \ldots\ldots (10)$$

using the author's notation r. This equation will be observed to be that commonly employed in American practice for vertical stirrups.* At rupture of the concrete beam, f_s will assume the value of the yield strength of the steel f_y.

In the case of the load at a distance equal to or less than 1.5 h, it would seem consistent with the author's photographs to assume a possible rupture plane as extending from the load point to the point of support. For such a plane the number of stirrups which crosses it will equal a/s. And consequently, total shearing force on the section and maximum shearing stress will have the following values.

$$V = \frac{A_w f_s a}{s}$$

$$v_{max} = \frac{a A_w f_s}{sb\ jd} \quad \ldots\ldots (11)$$

Observing that jd may be reasonably well approximated by 0.8 h and that at rupture f_s becomes f_y, it follows that at rupture

$$v''_{max} = 1.25\, C \left(\frac{A_w}{sb}\right) f_y \left(\frac{a}{h}\right) = D\, r f_y \left(\frac{a}{h}\right) \quad \ldots\ldots (12)$$

where the constant D replaces the constant product 1.25c. The constant D reflects the uncertainty in the nature of the rupture plane and must be chosen such that Eq. 11 and Eq. 12 will yield the same value for v_{max} at the common point $a = 1.5$. Simply equating Eq. 11 and 12 for $a = 1.5\ h$, one finds that the constant D has the value 2/3. Substituting this into Eq. 12 and subsequently adding it and also Eq. 11 in turn to Eq. 9 there results the complete expressions for equivalent shearing stress at rupture. Using the author's experimental data, it will be simple to verify that a good value for the constant B is 4. The final form of the equations then is

$$a \geqq 1.5\ h \qquad v''_{max} = \left(\frac{h}{a}\right)(0.100\, f_c' + 7500\, p) + r f_y$$

$$a \leqq 1.5\ h \qquad v''_{max} = \left(\frac{h}{a}\right)(0.100\, f_c' + 7500\, p) + \left(\frac{2a}{3h}\right) r f_y \quad \ldots\ldots (13)$$

It is not claimed that for the author's data these equations yield better results than his empirical equation (although it will be seen that the results are equally as good). It is claimed, however, that the above set of equations is more general and *when due recognition is given to the assumptions upon which it is founded*, it may be used to predict the shear capacity of a beam with some confidence. It should also be remarked here that the constant B, since it was found from the experimental data, includes any correction which would normally be applied to account for the presence of the normal stresses discussed in connection with Magnel's work.

Table A shows the agreement between Eq. 13, the author's expression, and results found from his tests. A value of $f_y = 50{,}000$ was used throughout since it represents a fairly good average of values in the author's Table 1.

*Peabody, D., Jr., *Reinforced Concrete Structures*, John Wiley and Sons, 2nd Edition, 1946, pp. 56-57.

TABLE A—COMPARISON OF RESULTS

Beam No.	Experimental value v_u	Value from Eq. 13 v'_{max}	$\frac{v'_{max}}{v_u}$	Author's value v_c	$\frac{v_c}{v_u}$
A1- 1	489	486	0.994	554	1.133
2	461	478	1.037	547	1.187
3	491	476	0.969	545	1.110
4	538	486	0.903	555	1.032
B1- 1	615	528	0.859	577	0.940
2	564	546	0.968	595	1.055
3	628	531	0.846	580	0.924
4	591	528	0.893	577	0.976
5	531	539	1.015	589	1.109
B2- 1	665	707	1.063	638	0.959
2	705	734	1.041	665	0.943
3	736	723	0.982	653	0.887
B6- 1	806	691	0.857	745	0.924
C1- 1	594	546	0.919	577	0.971
2	664	554	0.834	584	0.880
3	528	528	1.000	558	1.057
4	606	583	0.962	614	0.982
C2- 1	623	680	1.091	616	0.989
2	644	695	1.079	631	0.980
3	695	686	0.987	622	0.895
4	614	716	1.166	653	1.064
C3- 1	497	420	0.845	447	0.899
2	446	417	0.935	444	0.996
3	418	419	1.002	446	1.067
C4- 1	681	592	0.869	636	0.934
C6- 2	898	818	0.911	867	0.965
3	921	812	0.882	861	0.935
4	903	843	0.934	894	0.990
D1- 1	634	655	1.033	673	1.062
2	751	654	0.871	672	0.895
3	542	631	1.164	649	1.197
D2- 1	613	673	1.098	666	1.086
2	657	701	1.067	694	1.056
3	706	685	0.970	678	0.960
4	708	680	0.960	673	0.951
D3- 1	847	899	1.061	830	0.980
D4- 1	662	864	1.305	734	1.109
D1- 6	636	641	1.008	657	1.033
7	651	644	0.989	660	1.014
8	676	642	0.948	658	0.973
E1- 2	802	781	0.974	713	0.889
D2- 6	610	647	1.060	646	1.059
7	572	639	1.117	639	1.117
8	615	624	1.015	622	1.011
D4- 1	614	572	0.932	611	0.995
2	575	559	0.972	599	1.042
3	610	533	0.874	573	0.939
D5- 1	532	514	0.966	590	1.109
2	570	524	0.919	600	1.053
3	573	510	0.890	586	1.023
A0- 1	184	193	1.049	229	1.245
2	221	225	1.018	262	1.186
3	245	209	0.853	246	1.004
B0- 1	249	249	1.000	279	1.120
2	194	252	1.299	282	1.454
3	264	249	0.943	278	1.053
C0- 1	358	324	0.905	344	0.961
2	366	311	0.850	330	0.902
3	344	312	0.907	332	0.965
D0- 1	454	449	0.989	453	0.998
2	533	454	0.952	458	0.859
3	457	450	0.985	455	0.996

By ROBERT S. ROWE*

The test results reported by Mr. Clark are of interest; the photographs of typical beam cracks at failure are splendid. It is probably an oversight that the ultimate loads were omitted from Table 2 so that it is not possible to check the computed values of v_u. One may question whether it is profitable to develop a new empirical formula and then compare it with an already existing empirical formula.

It has been long known that cracking and structural behavior that precedes failure in reinforced concrete beams are complicated.† It does not seem quite correct to state that failure is due to this or that cause, as failure is usually due to a combination of causes. It is of some interest to note that as early as 1907,‡ tests show that the amount of horizontal steel has a direct bearing on what has been referred to as shear failure and that average shearing stress increases as the loads are positioned nearer the supports.

It seems desirable to ask why the tests results were not correlated with the usual beam theory as applied to reinforced concrete structures. A theoretical investigation would have shown that the load carrying capacity of a beam, as far as diagonal tension is concerned, is dependent upon many factors, including shearing force, bending moment, properties of the section and material, and amount of steel.

The bending moment is an important factor in diagonal tension and the shearing force cannot be treated separately as principal stresses are a function of both shearing force and bending moment. For a given principal stress, higher loads are permitted when the loads are moved closer to the supports, because the bending moment and stresses are smaller. One may question whether it would not be more profitable to consider the load-carrying capacity of a beam, as far as diagonal tension is concerned, as a function of the principal stresses, which include both the bending moment and the shearing force, rather than a function of the shearing force alone.

It seems proper to request a brief description of how the strain gages were employed, if any difficulties were encountered, and what limitations were found.

AUTHOR'S CLOSURE

The discussions present interesting comments and analyses which are helpful in the study of diagonal tension. The intelligent discussions are appreciated for their value and also as a confirmation of the thought that this is a subject of wide interest.

A considerable amount of detail of the tests and the study of the data was necessarily omitted from the paper due to space limitations. Contrary to the assumption in one discussion, an effort was made to derive a formula for general application which could be justified by the results of these and data from previous tests by others. However, it was evident that additional data

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†Turneaure, F. E., and Maurer, E. R., *Principles of Reinforced Concrete Structures*, John Wiley and Sons, Inc., New York, N. Y., 1907, p. 113.
‡*Ibid*, pp. 140-141.

TABLE B—RESULTS AVERAGED FOR GROUPS OF SIMILAR BEAMS WHICH FAILED IN DIAGONAL TENSION

Beams	f'_c, psi	Tensile reinforcement		Stirrups		j	a, in.	$\frac{d}{a}$	P, lb	v_u, psi
		Pieces and bar No.	p	s, in.	r					
8 x 18-in. beams—6-ft span*										
A1	3500	3- #10	0.0310	7.2	0.0038	0.830	36	0.427	101,032	495
B1	3490	3- #10	0.0310	7.5	0.0037	0.830	30	0.512	119,585	586
B2	3600	3- #10	0.0310	3.75	0.0073	0.832	30	0.512	143,600	702
B6	6110	3- #10	0.0310	7.5	0.0037	0.860	30	0.512	170,561	806
C1	3805	2- #10	0.0207	8.0	0.0034	0.856	24	0.640	125,966	608
C2	3655	2- #10	0.0207	4.0	0.0069	0.854	24	0.640	131,767	627
C3	2020	2- #10	0.0207	8.0	0.0034	0.821	24	0.640	91,728	454
C4	3550	3- #10	0.0310	8.0	0.0034	0.830	24	0.640	139,061	681
C6	6650	3- #10	0.0310	8.0	0.0034	0.864	24	0.640	192,946	907
D1	3680	2- #9	0.0163	6.0	0.0046	0.867	18	0.854	125,370	588
D2	3615	2- #9	0.0163	4.5	0.0061	0.866	18	0.854	135,370	635
D3	4090	3- #9	0.0244	3.0	0.0092	0.852	18	0.854	177,561	847
D4	3350	2- #9	0.0163	2.25	0.0122	0.862	18	0.854	140,370	662
8 x 18 in. beams—6-ft span—no web reinforcement										
A0	3445	2- #7	0.0098	—	—	0.888	36	0.427	44,282	203
B0	3435	2- #7	0.0098	—	—	0.887	30	0.512	51,450	236
C0	3500	2- #7	0.0098	—	—	0.888	24	0.640	76,715	351
D0	3755	2- #7	0.0098	—	—	0.892	18	0.854	99,995	456
6 x 15-in. beams—8-ft span†										
D1	4033	2- #10	0.0342	8.0	0.0046	0.833	24	0.515	80,894	654
6 x 15-in. beams—9 ft 7 in. span										
E1	4375	2- #10	0.0342	5.0	0.0073	0.837	25	0.495	99,717	802
6 x 15-in. beams—10-ft span										
D2	4063	2- #10	0.0342	6.0	0.0061	0.833	30	0.412	74,050	599
D4	3630	2- #10	0.0342	7.5	0.0049	0.826	30	0.412	73,550	600
D5	4053	2- #10	0.0342	10.0	0.0037	0.833	30	0.412	69,050	558

*For 8 x 18 in. beams, d = 15.37 in.
†For 6 x 15-in. beams, d = 12.37 in.

were needed to establish the influence of some of the factors. The empirical formula, as pointed out, was not intended for general application but to indicate the factors which must be considered. The statement in the synopsis that the position of the loads influences the shear capacity had not been previously demonstrated had reference to test data.

The SR-4 strain gages were cemented to the reinforcing bars and protected generally by the procedure recommended by the manufacturer. As a rule 18 out of 20 gages used in each beam gave, in general, results but unfortunately the failure of the two made results of little value in some cases.

Table B gives values of d and d/a and other pertinent data averaged for groups of similar specimens. These data are based on the data presented for individual specimens in Table 2.

Title No. 48-12

Coral and Salt Water as Concrete Materials*

By JOHN G. DEMPSEY†

SYNOPSIS

Concrete in which coral and its related soft and porous limestones are used as aggregate presents more than the usual number of problems in controlling uniformity and quality. Using sea water for mixing has always been a reluctant last resort. This paper describes coral materials, the problems arising from their use, and the methods of mix control used to obtain uniformly satisfactory concrete. Notes on the action of sea water are presented with an account of field experience with it.

CORAL AGGREGATE CHARACTERISTICS

When construction of military bases in the Bermuda Islands was undertaken, local deposits of coral and aeolian limestone were used for concrete aggregate. The problems encountered and the methods used to solve them can best be visualized when the nature of the rock itself is pictured.

The rock on a coral island is a limestone in its earliest stages of formation and thus varies widely in density and surface characteristics, as may be seen by a brief glance at its history. Laid down on a comparatively shallow bottom, the material undergoes several changes. The coral polyp or the inhabitant of the shell dies, and the skeleton fills with water which gradually becomes a saturated solution of calcium carbonate. If the shell fills with sand, the grains are cemented together by the saturated solution. In the absence of sand, deposits of calcium carbonate form. Sometimes the water deepens and pressure and water harden the material, the water dissolving the softer parts, leaving many holes and crevices.

Aggregate dredged from the sea is thus likely to be rough in surface and irregular in shape. It is characterized by a high percentage of long, thin particles about 60 percent of which are coral fragments with 10 to 20 percent shells. It is quite tough, with the specific gravity of coarse aggregate close to 2.45 and 2.53 for sand sizes on a saturated surface dry basis. Fig. 1 and 2 shows the general appearance of the more common particle types.

Where old bottoms became exposed, winds laid down this material in dunes, blowing away the extreme fines. Percolating water and oxygen in the air later hardened the dune surfaces, at the same time dissolving out and redepositing the more soluble material, so that cavities from microscopic size up to extremely large volumes resulted. As a consequence the native rock,

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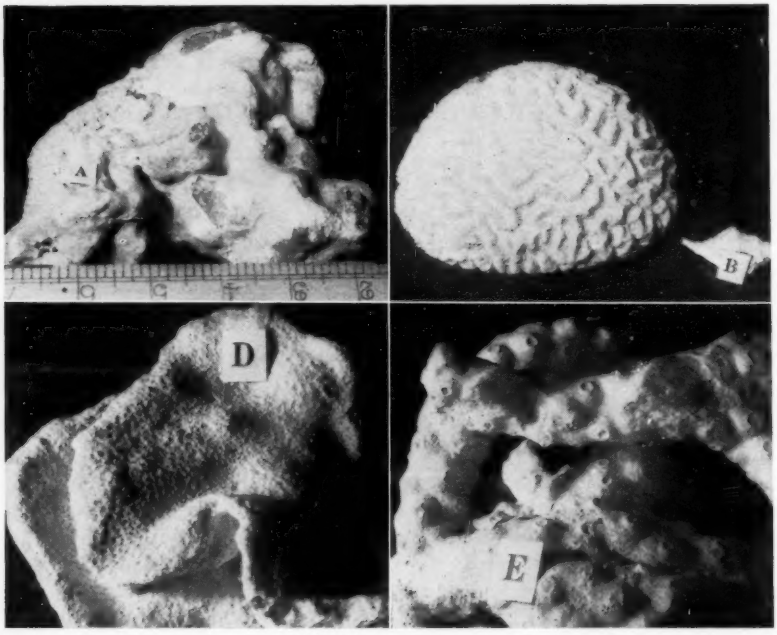

Fig. 1—Coral aggregate materials. Top left—Crystalline material dredged from sea bottom. Bottom left—Star coral dredged from sea bottom. Irregularity and extreme roughness are characteristic. Specimens range in size from 2 in. to 2 ft or more. They are tough and hard, with very large surface area. Top right—Brain coral common in dredged aggregate. Bottom right—Tree coral common in dredged material. Fragments which appear in the mix are about the size of small twigs.

while not very hard, is extremely rough, sharp, and irregular. Fig. 3 and 4 shows the character of the rock, while Fig. 5 shows the extremely rough and irregular shape of the sand sizes.

Aggregate quarried from these deposits of aeolian limestone or old dunes

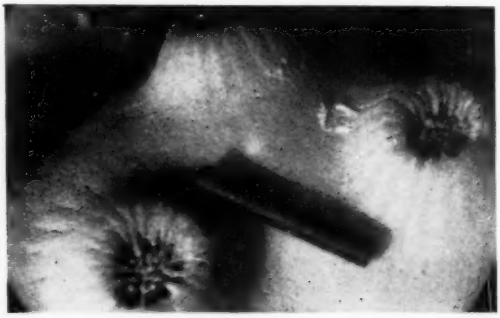

Fig. 2—Tree coral 4X natural size shows surface characteristics

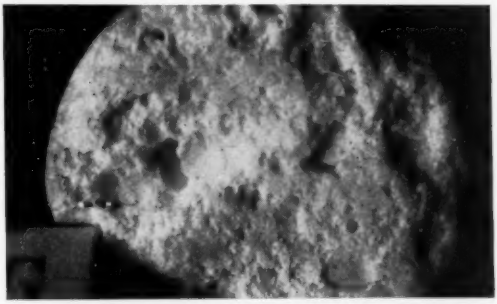

Fig. 3—Rock core of aeolian limestone deposit. Note cavities and irregular texture

Fig. 4—Coarse coral aggregate

showed very sandy, soft, surface texture. The hardest rock occurs usually in the 3 ft nearest the surface of any outcrop and around the roofs and walls of caves. All of it, after crushing, is irregular in shape with high surface areas and has a high percentage of fine material adhering to its surfaces, and very rough texture. The specific gravity of coarse sizes varies between 1.90 and 2.28. The more porous rock, which sometimes runs as high as 25 percent absorption by weight, breaks down into sand on crushing and absorption of crushed coarse aggregate averages between four and 13 percent. Sand from the crushing operations, while light in weight with an apparent specific

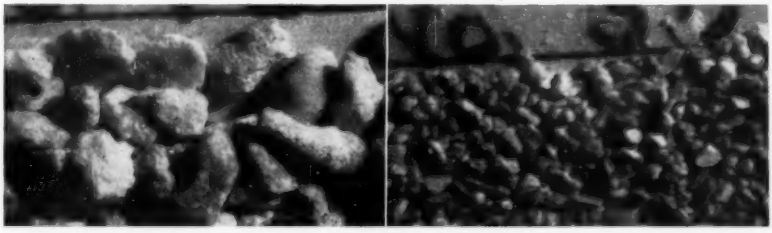

Fig. 5—Left—Coarse coral sand, retained on No. 10 sieve. Right—Fine coral sand, retained on No. 30 sieve

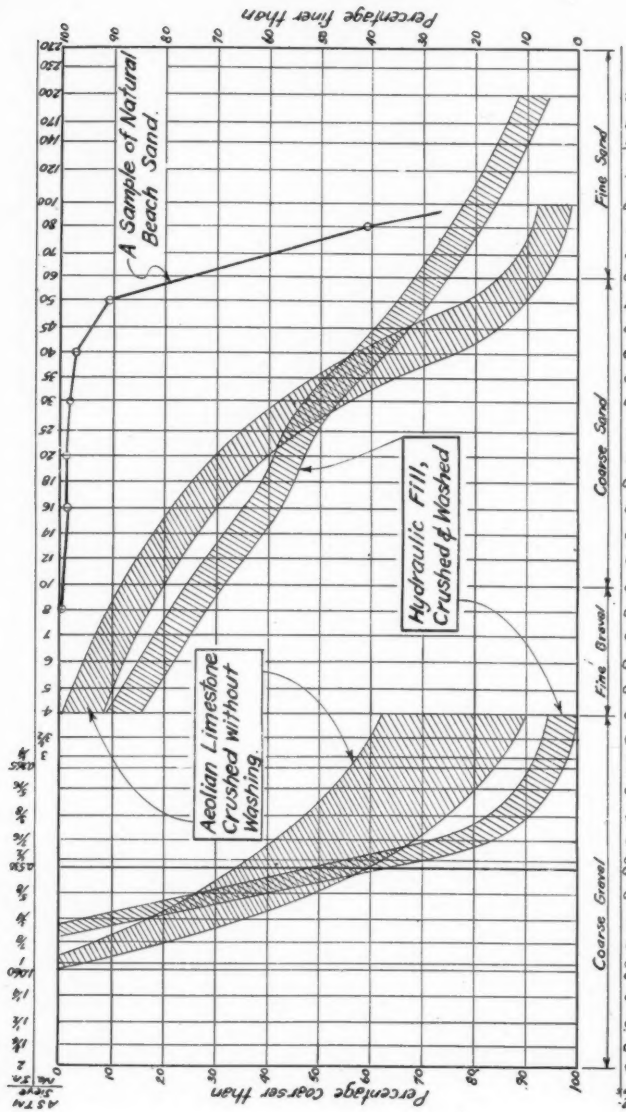

Fig. 6—Gradation curves for aeolian limestone and hydraulic fill materials. Width of bands show deviation from average for different sizes

gravity of 2.56 when saturated surface dry, showed quite good grading and absorption of about 1.6 percent by weight.

Fig. 6 shows the grading obtained on both quarried and dredged materials. The narrow band representing the curves of the dredged material shows the high uniformity obtained by washing through classifying screens. The still narrow band showing quarried sand reflects the essentially uniform grading of the particles loosely cemented originally into aeolian limestone. The 28 percent variation in material passing the No. 4 sieve showing on the quarried coarse aggregate reflects a special problem.

The quarried aggregate was screened dry. Moisture conditions in both air and material varied widely, causing variations in the amount of fine material adhering to the rough surfaces of the coarse sizes and carried through with them into the coarse aggregate hopper. This problem was handled by designing the mix for perfect separation on the No. 4 sieve, and computing the scale weights to achieve this at the batcher.

SALT WATER FOR MIXING

The only source of drinking water in Bermuda is rain water, since supplemented to some extent by evaporation. Its scarcity was such that none was available for construction and sea water was used for mixing concrete, for washing aggregate, or for both. In some instances very brackish well water was available and it was used in conjunction with washed aggregate. The resulting concentration of salt was about the same in either case.

The principal effect of the salt water on fresh concrete was to increase the steepness of the early part of the strength-gain curve so that the percent of 28-day strength normally obtained in seven days was obtained in three. This accelerating effect gradually diminished until the 28-day strength appeared little different than if the mixes had contained fresh water. Whatever effect it may have had in increasing the rapidity of stiffening of the fresh mix could not be distinguished from the effects of high temperature and rapid evaporation.

Another question will occur to the reader as it did to the staff on the job, and that is the long-range effect of sea water on the durability of the concrete. U. S. Navy *Bulletin* No. 36 reports a series of tests including 1:2 and 1:3 mortars mixed with salt water and stored for seven years in wood boxes and 16 years eight months in tidewater. The general condition of the salt water specimens was reported as excellent, with some erosion at corners and steel with 1 in. cover in one instance coated with a thin coat of rust. In none of the samples was the metal reinforcement pitted.

It is evident that on the evaporation of the mixing water the salts in solution are left behind in the concrete. In the case of sea water, these salts include sodium chloride and magnesium sulfate. These salts are inert as to either the set cement or the reinforcement except in the presence of air and moisture.

In porous concrete it is possible for water vapor in the air to condense in the capillary passages and voids under and around the coarser aggregate particles, bringing the salts originally present into solution which may attack both cement and steel. Where two different metals are present in the mass this solution becomes an electrolyte, and accelerated corrosion proceeds.

Since concrete is not attacked by corrosive solutions unless they can penetrate into the mass either by contact or by condensation and solution of residual salts, it is plain that a dense concrete of low porosity must be obtained. The production of this dense concrete was the principal goal toward which analysis and mix control was directed.

The first step toward insuring maximum density of the concrete was the inclusion of fine material passing the 100- and 200-mesh sieves. As high as 12 percent of sand passing No. 200 was permitted, although generally this fraction was held below eight percent. Davis and Troxell* report that very fine material of crystalline character does not produce marked increases in shrinkage, and this proved to be the case. In addition, absorption of concrete after 28 days was reduced from 13 to 10 and in some cases to eight percent by weight.

It was also found that a proprietary admixture containing calcium lignosulfonate permitted a 15 percent reduction in water without loss in strength or workability. The adoption of this material further increased the density of the concrete, through retaining a higher percentage of solids and increasing cohesiveness and plasticity.

During the first four years' service no evidence of excessive corrosion or spalling appeared which might have indicated that concrete mixed with sea water would have a less satisfactory service life than if it had been mixed with fresh water.

MIX DESIGN AND CONTROL

Since the majority of structures were of moderately light section and generously reinforced, 1 in. was selected as the maximum size of coarse aggregate. As most of the pours were not vibrated, 6 in. was decided on as the working slump for a general purpose mix. Preliminary studies showed that for this type of mix the optimum sand content was 42 percent of the absolute volume of total aggregate, and that for a 6-in. slump 42.5 gallons of water per cu yd was required.

The first mixes were almost unmanageable due to rapid stiffening and at the start this was attributed to the sea water being used for mixing and the high temperature of the fresh concrete due to the hot sun beating on the mixer drum. However, when the aggregate was tested for the amount of water absorbed during mixing and handling, this was found to be from eight to 11 percent of the weight of the coarse aggregate. When the corresponding

*Davis, Raymond E. and Troxell, C. E., "Volumetric Changes in Portland Cement Mortars and Concretes," ACI *Proc.* V. 25, 1929, p. 210.

TABLE 1—LABORATORY LOG—APRIL 13 TO 15 AND MAY 13 TO 16

Test	Apr. 13		Apr. 14		Apr. 15			May 13		May 14		May 15		May 16	
	9 AM	Noon	9 AM	Noon	9 AM	Noon	5 PM	9 AM	Noon	9 AM	Noon	9 AM	Noon	9 AM	Noon
Sand															
Moisture, percent	5.5	5.5	5.9	5.6	5.5	5.5	5.9	7.0	7.5	7.4	7.4	8.2	7.8	9.3	8.9
Retained on No. 4, percent	10.5	7.5	11.0	8.5	10.8	9.5	14.0	19.5	13.0	14.0	25.0	12.5	18.0	20.9	21.5
Sand in coarse aggregate, percent	6.8	18.0	18.1	13.0	20.5	19.3	15.5	5.2	16.3	16.7	18.0	30.0	24.1	19.6	14.5
Fineness modulus, Tyler series	3.17		3.05		3.11			3.05		3.36		3.33		2.90	
Specific gravity, saturated surface dry	2.58		2.58		2.58			2.56		2.56		2.56		2.56	
Absorption, percent	0.5		1.1		1.0			1.4		1.6		0.7		1.2	
Coarse aggregates, 1-in. max.															
Fineness modulus, Tyler series	6.56		5.79		5.72			6.34		6.00		5.45		5.83	
Specific gravity, saturated surface dry	2.11		2.16		2.08			2.13		2.14		2.22		2.26	
Absorption, percent	6.7		6.7		7.1			11.8		7.5		6.5		8.5	

amount of water was added to the mix, slump and consistency stabilized and no more difficulty was experienced in placing at the forms.

It was soon evident that not only was absorption very variable, but also particle soundness, apparent specific gravity, and grading, and all would require constant checking. There was a good deal of fine material added to the mix from grinding action due to softness of the coarser particles. Measurements made at different times indicated an average abrasion product of about 3.5 percent of the weight of coarse aggregate in the mix. It was decided that in the interest of density of the hardened concrete the additional fines were welcome rather than otherwise.

To permit control of the mixes it was decided that in addition to surface moisture tests, particle separation of both sand and stone on the No. 4 sieve would be measured each morning and noon and even oftener as circumstances required. In addition, specific gravity and absorption and sieve analyses were made daily. Specific gravity of sand was not measured as frequently when it was found to vary not more than 0.02 from its average value of 2.56. An idea of the mix control problem can be had from an inspection of Table 1, which is the laboratory log for periods in April and May.

To cope with such wide variations, adjustment of the scales at the batch plant for moisture content and to hold constant both size separation and yield was made following each test. It was found that not over one percent error was introduced on the No. 4 size separation check if it was made with damp sand. This saving in time made it possible to keep adjustments close to the batching schedule. The test for moisture (Chapman flask method), size separation test, computations necessary, and setting batch scales, took from 20 to 25 minutes, including time for sampling.

One of the most convenient tools for establishing control of the many variables encountered proved to be the study of summaries of test groups through their standard deviations and the percentage values of these, the

TABLE 2—SPECIFIC GRAVITY AND ABSORPTION TESTS

Number of tests	Max.	Min.	Ave.	Coefficient of variation, percent
		Specific gravity, coarse aggregate, 1-in. max. size		
27	2.26	2.04	2.15	2.7
26	2.24	1.92	2.16	3.4
26	2.25	2.08	2.18	2.1
26	2.28	1.88	2.13	4.1
Ave.			2.16	3.1
		Absorption, coarse aggregate, percent of saturated surface dry wt.		
25	12.50	4.30	7.71	29.7
		Absorption, sand, percent of saturated surface dry wt.		
25	3.00	0.30	1.58	39.2

coefficients of variation. These furnished means of studying concrete uniformity and the accuracy with which we were able to meet specification requirements. The standard deviation defines the area of normal job performance* and saves time otherwise lost in attempting to analyze variants from operation norms which are intrinsic in the operation and control methods in use. At the same time, their study helps focus attention on the points where lack of uniformity is most serious so that corrective measures can be applied with maximum effectiveness.

A summary of four months of specific gravity tests and a group of 25 absorption tests in Table 2 show the magnitude of the variation in density. The results obtained during the first six months as measured by 28-day compressive strength are shown in Table 3 along with the control range as established by this group of tests and the limit of uncertainty as to strength which they establish. The general summary in Table 4 shows the results obtained for the job with a brief sketch of background on each period covered.

TABLE 3—28-DAY COMPRESSIVE STRENGTH, GROUPS OF SEVEN, CHRONOLOGICAL ORDER

Group letter	Cement factor*	Compressive strength, psi			Coefficient of variation, percent
		Max.	Min.	Ave.	
A	7.00	3350	1820	2491	20.4
B	7.00	3100	2490	2803	11.3
C	7.00	3960	2180	3123	18.1
D	7.00	3120	2580	2829	7.6
E	6.76	4175	3370	3714	7.2
F	6.76	3780	2480	3249	11.6
G	6.76	3680	2340	2934	15.8
H	6.06	3180	2300	2683	12.4
I	6.06	3090	2130	2767	11.5
J	6.06	3960	2900	3383	11.0
Ave.					12.7

*Sacks per cu yd

Job control limits for 3000 psi mix: ± 500 psi for groups of seven tests. Limits of uncertainty: Nine times out of ten the average of seven tests will fall between 2700 and 3300 psi.

Concrete Manual, U. S. Bureau of Reclamation, 3rd Ed., pp. 180-194.

TABLE 4—JOB SUMMARY

Period	Cement factor, sacks per cu yd		Compressive strength, psi		Standard deviation, psi	Remarks
	3000 psi	4000 psi	7-day	28-day		
First quarter	7.00			2790	± 520	Aggregate a new and untried material, variable and poor in quality. High absorption during mixing and placing made mixes difficult to handle. Consistency satisfactory after compensating for water taken up by absorption.
Second quarter	6.76		2800		± 410	Boulders for aggregate manufacture picked from quarry excavation individually for best quality.
				3390	± 460	
Third quarter	6.06			3177	± 450	Material for aggregate loaded at quarry by power shovel with only general inspection for quality. Stone good to fair.
Job average after adjustment as in third quarter	5.85			3500	± 490	Aggregate handling as in third quarter. Proprietary admixture used permitting reduction in water without loss of workability.
Special pour for reservoir tanks		7.00		4510	± 490	Slump restricted to 4 in. Admixture for cement dispersion and water reduction added.

Briefly stated, the data indicate that the experience gained on the job made possible an increase of 50 percent in the strength of the initial 7-sack mixes to the final level of 4500 psi, and that with the materials at hand it proved possible to obtain 28-day compressive strength averaging 3500 psi using a cement factor of 5.85 sacks per cu yd of concrete.

CONCLUSIONS

1. Coral and its related aeolian limestone are suitable materials for concrete aggregate. With aggregate of 1-in. maximum size about 39 gallons of water per cu yd is required to give a 3-in. slump. Because of its great porosity care must be exercised to prevent absorption during the mixing or placing cycle.

2. Due to variations in density and soundness and the amount of No. 4 sizes carried over to coarse aggregate hoppers during dry screening, a systematic program of tests for gradation, size separation, specific gravity, and absorption is imperative if reasonable uniformity in the concrete is to be attained.

3. The standard deviation and information derived from it is of considerable value in analyzing and controlling factors affecting mix uniformity.

4. A workability agent permitting reduction in total water required for mixing without loss in consistency or strength helps to overcome internal friction in the fresh concrete mix and promotes density of the hardened concrete by keeping its liquid content at a minimum.

5. Sea water seems to be satisfactory for making reinforced concrete and develops no problem beyond an acceleration in stiffening of the fresh mix. There is no harmful effect on the durability of reinforced concrete insofar as observations during four years subsequent to completion of this work can determine. However, due to the corrosive action of any residual salts in the presence of air and moisture, care in developing maximum density in the set concrete is of paramount importance if satisfactory durability is to be assured.

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Discussion of a paper by John G. Dempsey:

Coral and Salt Water as Concrete Materials*

By E. W. SCRIPTURE, JR., and AUTHOR

By E. W. SCRIPTURE, JR.†

Mr. Dempsey has made an interesting contribution to a subject concerning which little has been published. Particularly valuable are his data on the use of salt water for mixing, a question that has frequently been raised in the past. The information on the use of soft and variable aggregates should be of value in connection with the construction program being carried on in various parts of the world.

It is difficult to reconcile the author's statement on p. 161, in speaking of salt water for mixing, "Whatever effect it may have had in increasing the rapidity of stiffening of the fresh mix could not be distinguished from the effects of high temperature and rapid evaporation," with his conclusion number 5, in which he states: "Sea water seems to be satisfactory for making reinforced concrete and develops no problem beyond an acceleration in stiffening of the fresh mix."

On p. 162 the author states: "It was also found that a proprietary admixture containing calcium lignosulfonate permitted a 15 percent reduction in water without loss in strength or workability." He makes a similar statement concerning the effect of this admixture on strength in his conclusion 4.

In Table 4, however, the author shows that a mix with a cement factor of 5.85 sacks per cu yd with the admixture gives a 28-day strength of 3500 psi, while a mix which appears to be made with the same materials without the admixture, with a cement factor of 6.06 sacks per cu yd, gives a strength of 3177 psi. In other words, with a lower cement factor the strength is increased with the admixture. The inference to be drawn from the author's data is not, therefore, that the admixture did not produce a loss in strength but that it produced an increase in strength. This is in accord with our experience over a wide range of test data, which show that this admixture permits a reduction in water-cement ratio, with no decrease in flowability, and an improvement in workability, and that the concrete with the reduced water follows the water-cement ratio law with respect to strength, or stated another way it gives an increase in strength of 15 to 25 percent for a given cement factor.

*ACI JOURNAL, Oct. 1951, *Proc.* V. 48, p. 157. Disc. 48-12 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 24, No. 4, Dec. 1952, Part 2, *Proceedings* V. 48.

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AUTHOR'S CLOSURE

The question of premature stiffening of concrete mixes made with salt water in Bermuda was one that created a great deal of discussion among various members of the field staff. We were faced with three variables new to our experience. All of them might be considered to increase the stiffening of a fresh batch of concrete. First, the use of sea water for mixing; second, the use of unusually porous coral or soft limestone for aggregate; and finally the presence of a hot and powerful sun which beat on the drums of ready-mixed concrete equipment during hauling and the mixing cycle.

Later observation by the author of the behavior of carefully controlled mixes produced in still warmer climates seems to indicate that the salt mixing water was the main factor.

In Puerto Rico ambient temperatures range between 80 and 90 F. Here the first two variables might be said to be eliminated. Fresh water at ambient temperatures was used for mixing. In a large ready-mixed concrete plant the aggregates used were a graded river gravel of hard dense stone and well-shaped river sand of a specific gravity of about 2.65. Two other projects on the island were gravity dams using central plants from which the batch was dumped into open truck beds or buckets. One of the jobs handled a 4-cu yd concrete bucket by cableway and the other used open dump trucks from which the mix was subsequently discharged into a 3-cu yd bucket and handled by crane. On both dams an air-entraining agent was used. In none of these operations was premature stiffening ever a problem except when a fourth variable, hot cement, entered the picture on the ready-mixed concrete operation.

The same can be said of operations in Venezuela. Two projects in particular are worthy of mention. The first one was an aqueduct serving the city of Caracas from a new system of storage reservoirs. On this work the second variable, soft and porous aggregate, was a factor. The aggregates were of schistose materials. It was difficult to keep clean and well-graded, particle shapes were poor, and it was a constant problem to find sources from which sound aggregate particles could be produced. Ambient temperatures on this work were in the 80's generally. The second, on the northern coast, was the construction of large oil refineries on the Paraguaná Peninsula. Fresh water was used for mixing and aggregates were a porous limestone slightly superior in hardness to the Bermuda material but otherwise very similar. Sand was a beach sand which was also mostly limestone material often with a high percentage of sea shells. Ambient temperatures ranged from 90 to more than 100 F and a brilliant hot sun and driving wind was the rule in this area.

On neither job was premature stiffening a serious problem although it was mentioned occasionally as being somewhat of a factor in finishing floor or roof slabs. On these jobs workability or air-entraining agents were not generally used.

These later observations have caused the author to believe that the stiffening of the mix which was a problem in Bermuda was probably due to the effect of the salt water used in mixing. This is a matter of opinion in that

circumstances did not permit running tests which might have given a more conclusive answer. With respect to both stiffening and strength gain, comparative tests over a period of at least a year under carefully controlled conditions of identical mixes, one using salt water and the other fresh would certainly be of great value and interest.

The author cannot agree that the data submitted in Table 4 admits of the conclusion that the admixture used necessarily produced an increase in strength. It is quite plain that there was an increase which might be appropriate to the reduction in water reported. However, the reported reduction in cement is only 3½ percent and the increase in strength only 10 percent, a strength increase well inside the job coefficient of variation. Other factors could easily have entered into the situation. For example, the material quarried for aggregate was quite variable, as indicated in the specific gravity and absorption and their variation shown in Table 2. Another factor which caused considerable variation in compressive strength was the character of the project which included many different types of structures such as pavements, reservoirs, buildings of various sizes, and others. All these factors are of course reflected in the coefficient of variation.

It is certain that within the job control range of about 500 psi which job conditions imposed, it was possible to increase the minimum compressive strength of concrete made with coral and soft limestone aggregates from slightly in excess of 2700 psi to better than 3300 psi in 28 days using the control methods described and a cement content of about 6 sacks per cu yd.

To conclude, the experience in Bermuda hardly warrants any positive conclusion as to a strength gain that might be attributed to the use of the admixture because of the many other variables; especially those of changing absorption, fineness, water demand, and soundness of aggregate which was available. With regard to the use of sea water for mixing, it is true that during the work in Bermuda it was difficult to say which of several factors caused the problem of rapid stiffening. Later observations on work under similar climatic conditions indicated that sea water was indeed the main factor. In other respects the use of salt water, while perhaps calling for somewhat more care in following customary precautions during construction to insure high quality work, certainly presented no problem in either use or the results obtained.

Title No. 48-13

Load Carrying Capacity of Dowels at Transverse Pavement Joints*

By HENRI MARCUS†

SYNOPSIS

The load carrying capacity of dowels at transverse joints of concrete pavements is dependent upon width of joint, size and length of dowel, slab thickness, strength of concrete and steel, dowel alignment and compressibility of the subgrade supporting soil.

To evaluate the efficiency of the dowels, three test groups investigated the effect of various factors: (1) resistance of concrete to bearing stresses distributed uniformly by dowels of different diameters; (2) effect of dimensions and spacing of dowels, dimensions and strength of concrete and compressibility of subbase; and (3) sliding resistance of dowels coated with various substances.

INTRODUCTION

Round steel bars placed horizontally and parallel with the longitudinal axis of the slab are the load carrying device most frequently used at expansion joints in runway slabs. These dowels are embedded in concrete on one side of the joint; on the other side they are provided with expansion sleeves and coated with a thin layer of paint or grease to allow the bars to slide freely in the sleeve.

The load carrying capacity of this device is dependent upon width of joint, size and length of dowel, slab thickness, strength of concrete and steel, dowel alignment and compressibility of subgrade supporting soil.

Some of these factors were investigated by Westergaard,[1] Friberg,[2] Kushing and Fremont,[3] Giggin,[4] Teller,[5] and Finney and Fremont.[6] One of the most significant results of Friberg's investigations[2] is the disclosure of the extremely high bearing stresses acting on the concrete around the dowel at the face of the joint. He found, for instance, that a vertical force of 5000 lb applied on the dowel $\frac{3}{8}$ in. from the face of the joint will produce immediately below the dowel a bearing pressure on the concrete equal to 14,050 psi for a $\frac{3}{4}$-in. diameter dowel; 10,045 psi for a $\frac{7}{8}$-in. dowel; 8100 psi for a 1-in. dowel; and 5350 psi for a $1\frac{1}{4}$-in. dowel.

The bearing stress usually allowed for concrete having the conventional 28-day compressive strength f_c' is limited to $0.375\ f_c'$; for instance, the allow-

*Presented at the ACI Washington Regional Meeting, Washington, D. C., Oct. 24, 1950. Title No. 48-13 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 2, Oct. 1951, *Proceedings* V. 48. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Feb. 1, 1952. Address 18263 W. McNichols Rd., Detroit 19, Mich.

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TABLE 1—BEARING STRENGTH OF DOWELS IN 12-IN. CONCRETE BLOCKS

Specimen No.	Diameter of dowel, in. d	Compressive strength of concrete, psi f_c'	Bearing strength			$\frac{f_b}{f_c'}$	Manner of failure
			Total rupture load, R, lb	Load per unit of dowel length, $P = \frac{R}{l}$	Average bearing stress, psi, $f_b = \frac{R}{ld}$		
A1	¾	3740	89,000	7417	9890	2.64	*
A2		3880	84,000	7000	9330	2.40	†
A3		3640	93,700	7808	10400	2.86	
Ave.		3780	88,900	7408	9873	2.61	
B1	1	3950	106,500	8875	8875	2.25	*
B2		3880	110,000	9170	9170	2.36	*
B3		3730	108,000	9000	9000	2.41	*
Ave.		3850	108,200	9020	9020	2.34	
C1	1½	3530	115,000	9583	6390	1.81	*
C2		3680	118,500	9875	6580	1.79	*
C3		3390	115,000	9583	6390	1.88	*
Ave.		3530	116,200	9683	6450	1.83	
D1	2	3770	157,000	13083	6540	1.73	‡
D2		3680	159,500	13292	6650	1.81	*
D3		3390	145,000	12083	6040	1.78	*
Ave.		3610	153,800	12817	6410	1.78	

*Split vertically through center.
†Split vertically on both sides first, short diagonal cracks radiating from dowel appeared on one side.
‡Split vertically on both sides first and then diagonally on one side.

able stress for 3000-lb concrete is 1125 psi. Comparing this with the stresses found by Friberg, even the lowest pressure exerted by a 1¼-in. dowel far exceeds allowable limits.

Actually concrete is able to withstand a concentrated bearing stress many times greater than f_c' without being overstrained. The local bearing strength of concrete is dependent on many factors: dimensions of loaded area, depth of concrete below the dowel, and last but not least, shear and tensile strength of the concrete.

To determine the effect of these various factors, tests were performed for the Bureau of Yards and Docks by the National Bureau of Standards. One group of tests investigated the resistance of concrete to bearing stresses distributed uniformly by dowels of different diameters but equal lengths on concrete blocks of various depths. In the second group of tests the dowels were subjected to loads applied outside the concrete, producing a state of stress similar to that occurring in concrete sections immediately adjacent to expansion joints in runway slabs. These tests were designed to evaluate the effect of dimensions and spacing of dowels, dimensions and strength of concrete, compressibility of the base supporting the concrete blocks, and to find the extent that various types of reinforcement increase bearing strength. The third group of tests determined the sliding resistance of dowels coated with various substances.

This paper reviews briefly those results which are interesting not only for evaluation of the efficiency of these devices, but also for a better understanding of the more general problem of concrete strength.

RESISTANCE OF CONCRETE TO UNIFORMLY DISTRIBUTED BEARING STRESSES

Two series of tests were performed on 12 x 12-in. prismatic concrete blocks

TABLE 2—BEARING STRENGTH OF DOWELS

Specimen	Diameter of dowel, in.	f_c', psi	Bearing strength		Displacement of dowel, load of 5000 psi, in.
			Total load, kips	Load per sq in of projected area of dowel, psi	
In 6-in. concrete blocks					
6-A-1	¾	3720	119	13230	0.00060
6-A-2		3360	100	11110	0.00070
6-A-3		3250	100	11110	0.00130
Ave.		3440	106.3	11820	0.00870
6-B-1	1	4390	185	15420	0.00220
6-B-2		3890	163.5	13630	0.00215
6-B-3		3460	98.5	7910	0.00275
Ave.		3910	149	12320	0.00235
6-C-1	1½	3360	151.5	8420	0.00330
6-C-2		3430	154.5	8590	0.00405
6-C-3		3360	148.5	8260	0.00405
Ave.		3380	151.5	8480	0.00380
6-D-1	2	3580	160	6670	0.00560
6-D-2		3010	169	7050	0.00460
6-D-3		3400	190	7920	0.00400
Ave.		3330	173	7210	0.00473
In 18-in. concrete blocks					
18-A-1	¾	4000	109	12120	0.00160
18-A-2		3780	100	11110	0.00150
18-A-3		4250	90	10000	0.00250
Ave.		4010	99.7	11080	0.00187
18-B-1	1	4100	119	9920	—
18-B-2		4100	110	9100	0.00140
18-B-3		3980	120	10000	0.00150
Ave.		4030	116.3	9700	0.00145
18-C-1	1½	3460	130	7230	0.00270
18-C-2		4000	127.5	7090	0.00220
18-C-3		3700	143	7940	0.00290
Ave.		3720	133.5	7420	0.00260
18-D-1	2	4100	183.5	7650	0.00295
18-D-2		3400	150	6300	0.00300
18-D-3		4210	154	6420	0.00335
Ave.		3910	162.5	6790	0.00310

with depths of 6, 12 and 18 in. The loads were applied on ¾-, 1-, 1½- and 2-in. diameter steel dowels, embedded horizontally for one half their circumference in half circular grooves located at the center of the top surface of the blocks. The blocks were bedded in gypsum plaster directly on the lower platen of a 600,000-lb capacity hydraulic machine and the load was transmitted to the dowels over their entire length through a rigid block having a V-notch which rested directly on the dowels. The vertical strain was measured at each load increment by strain gages placed 2½ in. below the dowel.

Results of first test series

The results of tests performed on 12-in. deep cubic blocks are presented in Table 1. The tests disclose that the ratio of the bearing stress, f_b, to the allowable stress, $f_a = .375\ f_c'$, is 6.97 for ¾-in. dowels, 6.24 for 1-in. dowels, 4.87 for 1½-in. dowels and 4.73 for 2-in. dowels. The least of these is greater than the anticipated factor $f_c'/f_a = 2.67$.

TABLE 3—COMPARATIVE STRENGTH OF SPECIMENS WITH VARIOUS DEPTHS

Test series	Depth, in.	f_c', psi	Dowel diameter, in.	Average rupture load, R, lb	Comparative loads, lb $R_c' = R \times \frac{3780}{f_c'}$	$\frac{R_c'}{88{,}900}$
6A	6	3440	¾	106,300	116,806	1.314
12A	12	3780	¾	88,900	88,900	1.000
18A	18	4010	¾	99,700	93,982	1.057
					$R_c' = R \times \frac{3850}{f_c'}$	$\frac{R_c'}{108{,}200}$
6B	6	3910	1	149,000	146,713	1.356
12B	12	3850	1	108,200	108,200	1.000
18B	18	4030	1	116,300	111,105	1.027
					$R_c' = R \times \frac{3530}{f_c'}$	$\frac{R_c'}{116,200}$
6C	6	3380	1½	151,500	158,223	1.326
12C	12	3530	1½	116,200	116,200	1.000
18C	18	3720	1½	133,500	126,681	1.090
					$R_c' = R \times \frac{3610}{f_c'}$	$\frac{R_c'}{153,800}$
6D	6	3330	2	173,000	187,546	1.219
12D	12	3610	2	153,800	153,800	1.000
18D	18	3910	2	162,500	150,031	0.975

Results of second test series

Test results on 6- and 18-in. blocks are indicated in Table 2. To compare rupture loads R_6 and R_{18} of these specimens with rupture load R_{12} of the 12-in. deep specimens the following values were computed: $R_6' = R_6 \times f_6'/f_{12}'$ and $R_{18}' = R_{18} \times f_{18}'/f_{12}'$; f_6', f_{12}' and f_{18}' being respective compressive strengths of the 6, 12 and 18-in. deep specimens.

Comparative loads, R', are given in Table 3. The ratio R_6'/R_{12} varies according to dowel diameter between 1.219 and 1.366. The ratio R_{18}'/R_{18} between 0.975 and 1.090.

The shallow blocks seem to have greater bearing strength than the deeper specimens.

Analysis of test results

To understand the cause of variation in the ratio f_b/f_c' it is necessary to consider not only the one vertical stress component f_b but the whole state of stress and strain in the concrete around the dowel.

In fact, the pressure exerted on the dowel periphery is not vertical. It is radial pressure.

$$p_r = \frac{4}{\pi} \times \frac{R}{ld} \cos \phi \qquad (1)$$

ϕ being the angle between the direction of p_r and the vertical axis (Fig. 1). The resultant of the forces acting on one half of the block (Fig. 2) has a vertical component V and a horizontal component H. They are balanced: (1) by support reactions X, Y; and (2) by horizontal stresses acting on the vertical middle section CB and combined to a horizontal tensile force N and to a couple M.

The force X is statically indeterminate. It does not have the value $X = O$, as assumed in the conventional design of footings, nor the value $X = Y \tan \rho$ (ρ being the angle of friction between block and supporting base) be-

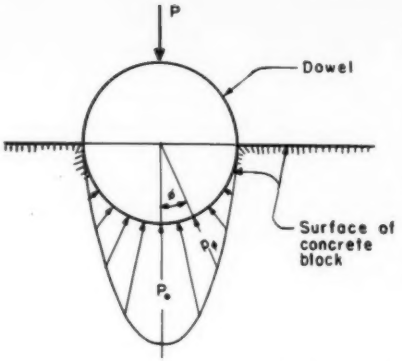

Fig. 1 (Above)—Pressure condition on periphery of loaded dowel

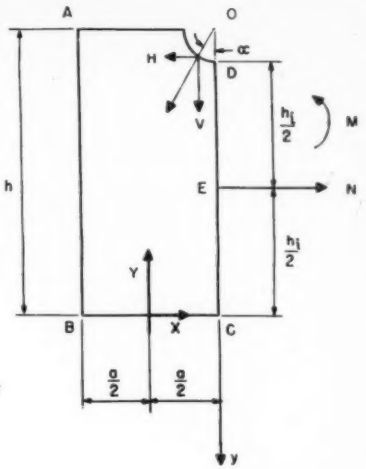

Fig. 2 (Right)—Stresses in half block

cause the horizontal displacements of the bottom particles of the blocks are too small to build up considerable frictional resistance.

A method of computing the reaction X in terms of the dowel load, R, the diameter, d, the half width, a, of the block, and the effective depth, $h_i = h - d/2$, has been developed.[7] Its application shows that X/Y is but a fraction of tan ρ. Knowing X and Y, it is possible to determine N and M, and also the horizontal tensile stress s_x acting at the top of the middle section and the vertical compressive stress s_y.

The horizontal strain resulting from the coaction of these two stresses is:

$$\epsilon_x = \frac{s_x + \mu s_y}{E} \qquad (2)$$

E being the modulus of elasticity of concrete, μ Poisson's coefficient. For $\mu = 0.15$, $E\,\epsilon_x = s_x + 0.15\,s_y = f_x$. The term f_x represents a fictitious stress producing a strain equivalent to the real strain ϵ_x. The values of s_x, s_y, f_x, corresponding to the rupture loads of the first series are given in Table 4.

While an increase in dowel diameter from ¾ to 2 in. decreases vertical stress s_y from 6262 to 4063 psi, the horizontal tensile stress s_y increases from

TABLE 4—STRESS AND STRAIN IN 12-IN. CONCRETE BLOCKS AT RUPTURE

Series	Rupture loads, R, lb	p_o, psi	s_x, psi	s_y, psi	f_x, psi	f_c', psi	f_x/f_c'
A	88,900	12577	381.5	6262	1320.8	3780	0.3494
B	108,200	11480	474.4	5717	1332.0	3850	0.3460
C	116,200	8219	532.4	4093	1146.4	3530	0.3248
D	153,800	8159	737.1	4063	1346.6	3610	0.3730

TABLE 5—MEASURED AND COMPUTED STRAINS

Concrete stress, s, psi	$f_c' = 3600$ psi		$f_c' = 5060$ psi	
	ϵ, in. per in.		ϵ, in. per in.	
	Measured	Computed	Measured	Computed
1200	.00039	.000379	.000335	.000305
1500	.00062	.000617	.00051	.000485
2400	.00088	.000885	.00069	.000685
3000	.00123	.001186	.00088	.000904

381.5 to 737.1 psi; however, the fictitious stress f_x and also the ratio f_x/f_c' remain *almost constant.*

In all cases, the specimens which have the same dowels but different depths have but slight variation in the critical values of f_x.

Critical strains

To determine the real strain corresponding to f_x the modulus of elasticity, E, must be known.

A series of auxiliary tests were performed to establish the effect of concrete strength, f_c', on the stress-strain relationship. An analysis of the results showed that the longitudinal strain, ϵ, produced by a longitudinal compressive strain, s, in concrete having the strength, f_c', has approximately the value

$$\epsilon = \frac{s}{f_c'} \times \frac{1 + \frac{3}{5}\frac{s}{f_c'}}{2000} \times \sqrt{\frac{f_c'}{1000}} \quad \ldots\ldots (3)$$

Table 5 gives the measured strain and the strains computed according to the formula for various stresses, s, and various strengths, f_c'. There is close agreement between observed and computed strains.

Applying Eq. (3) to the values $s = f_x$ for the first group of bearing tests (Table 4), ϵ for the specimens is: Series A—$\epsilon_a = 0.000411$; B—$\epsilon_b = 0.000410$; C—$\epsilon_c = 0.000365$; and D—$\epsilon_d = 0.000434$, or an average $\epsilon_x = 0.000405$. These results indicate that failure occurred when the horizontal elongation of concrete reached the critical value $\epsilon = 0.0004$.

The usefulness and reliability of the method used for figuring strains was confirmed by taking the vertical strain ϵ_y measured 2.5 in. below the dowel, computing the stresses s_x and s_y produced at that depth by a load $R = 80{,}000$ lb and also the corresponding fictitious stress, $f_y = E\,\epsilon_y = s_y + \mu s_x = s_y + 0.15 s_x$. By applying Eq. (3) to the stress $s = f_y$, values of ϵ_y were obtained. A comparison of these computed strains with the measured strains again discloses close agreement.

BEARING CAPACITY OF DOWELS LOADED OUTSIDE THE CONCRETE

To simulate the effect of a load transmitted by a dowel from one face to the other face of an expansion joint, a load P was applied vertically on a horizontal dowel of diameter d at e distance from the face of the model slab,

TABLE 6—ELASTIC PROPERTIES OF SUPPORTING BASE

Material of supporting base	Base thickness, in.	Length of concrete specimen overhanging supporting base, in.	Approximate modulus of compressibility, lb per in.^{3}, k_s
Wood	3¾	6	0
Cork	1½	0	100
Wood fiber	2¼	0	300
Wood	3¾	0	9000

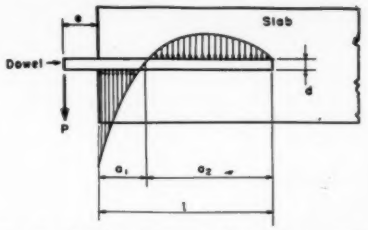

Fig. 3—Load diagram of dowel in concrete block

and carried into the slab by the dowel embedded in the concrete for a length l (Fig. 3).

The load is balanced by bearing stresses acting against the bottom of the dowel at its near end, and against its top at the far end. The pressures exerted on the concrete produce shear and bending stresses in sections parallel and normal to the dowel axis. Thus the concrete is subjected to tensile strains which affect the bearing capacity of the load-carrying device much more than do primary compressive strains in the bearing area.

It required both theoretical and experimental studies to determine the magnitude of these tensile strains and their relationship to the tensile strength of the concrete. The significant test results are reviewed briefly and the design procedure proposed for the evaluation of bearing capacity of dowels is described.

The samples were 20 in. long concrete blocks, between 6 and 12 in. deep and 12, 18 or 24 in. wide. Horizontal steel dowels, ¾ or 1 in. in diameter and embedded 6 to 10 diameters in the concrete, protruded from the two opposite end-faces at the center points of the blocks.

Materials

Concrete blocks—All samples, with one exception, were made of concrete proportioned 1:2.7:3.45 by weight with a cement content of 5.3 bags per cu yd and a water-cement ratio of 8 gal. per bag of cement. The concrete used for the odd specimen had a cement content of 7.1 bags per cu yd and 5.65 gal. of water per bag of cement. Potomac River sand and 1-in. gravel were used as aggregates.

Steel dowels—All dowels were made of cold-rolled steel rods, dipped in zinc chromate primer. They had a modulus of elasticity of 29,700,000 psi and a yield point of 79,000 psi for the ¾-in. bars and 67,000 psi for the 1-in. bars.

Reinforcement—Some specimens were reinforced by ⅜-in. and ½-in. deformed bars having a high bond value, others by cold-drawn ¼-in. wires having a high yield point, still others by 3/16-in. or ¼-in. cold-rolled steel face plates (Fig. 4).

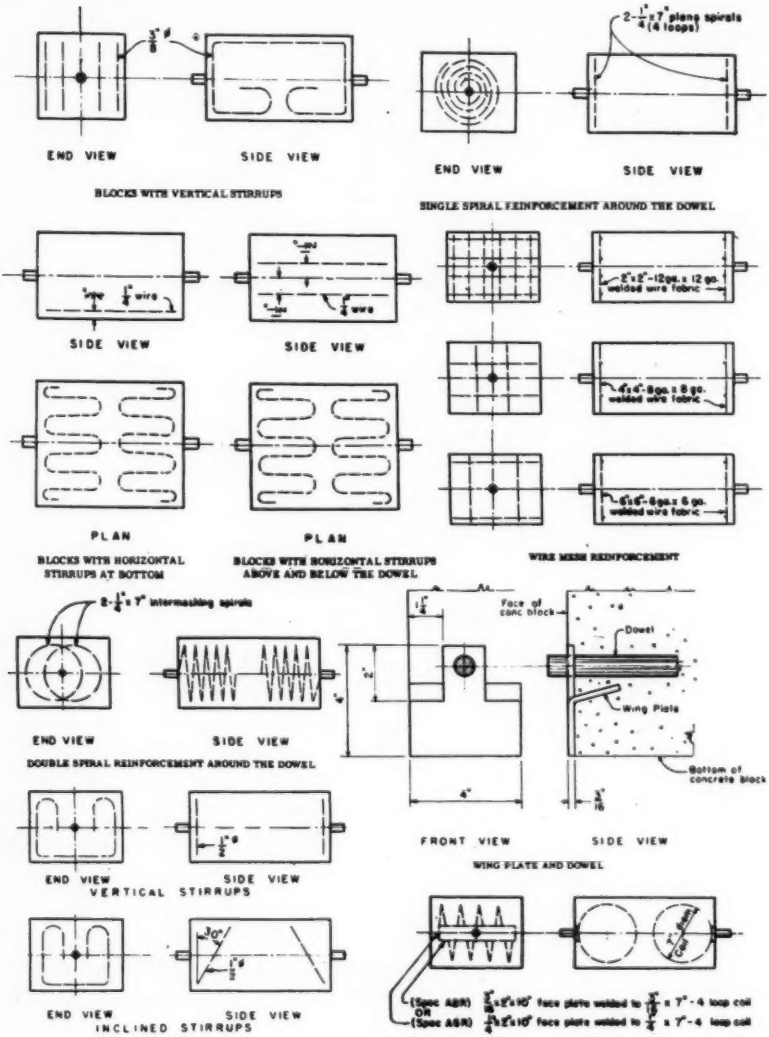

Fig. 4—Arrangement of reinforcement in test blocks

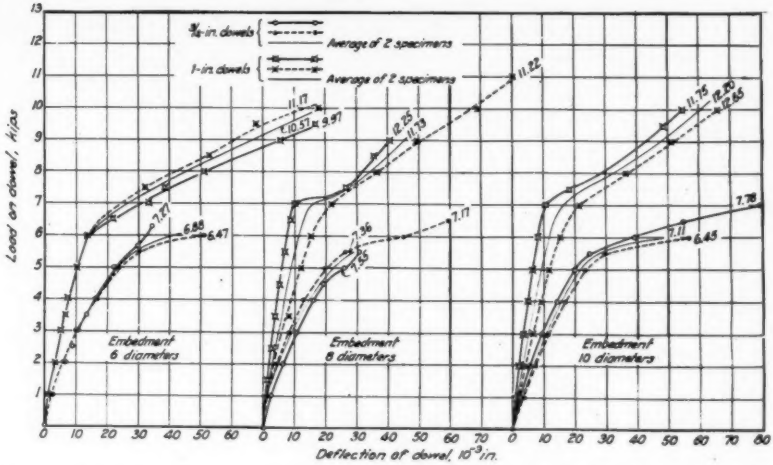

Fig. 5—Deflection of ¾- and 1-in. dowels tested in plain concrete blocks 8-in. deep supported on wood

Support conditions

The effectiveness of the subgrade as a support for a slab is dependent upon its compressibility. The samples were placed on four different bases having widely different elastic properties (Table 6).

Test procedure

The load was applied to the pair of dowels in each block by loading brackets attached to a 6-in. loading beam. The bearing edge of the brackets had semi-circular notches fitting over the dowel and centered on a point ⅜ in. from the faces of the concrete blocks. Deflections of the dowels at the face of the blocks were measured with 0.00001-in. micrometer dial gages supported on the top of the concrete blocks. The contraction of the "subgrade" was measured with two sets of three dial gages on each side, supported by rigid bases resting on the lower platen of the testing machine. Wire resistance strain gages were used for measuring compressive strain in the concrete.

Test results

Most significant results are illustrated in Fig. 5 and 6 which show the relationship between loads and dowel deflections for various support conditions, Fig. 7 and 8 showing the effect of the embedded length of dowels on load-carrying capacities and on dowel deflection, Fig. 9 and 10 showing the effect of depth of concrete blocks, and Fig. 11 and 12 showing the effect of width of block.

These data indicate that: (1) the crack load, P_c, is but slightly affected by dowel length if $l \geqq 8d$; (2) the crack load is almost proportional to concrete depth; (3) the modulus of compressibility of soft subgrades (k_s = 100 to 300 psi) does not affect greatly P_c or P_u if the samples are unreinforced, but the

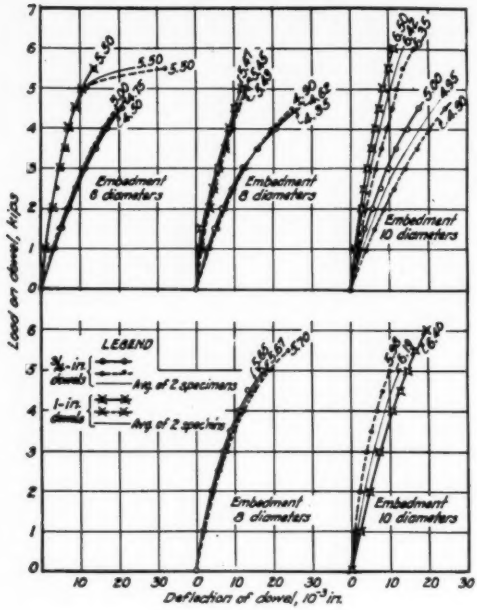

Fig. 6—Deflections of ¾- and 1-in. dowels tested in plain concrete blocks 8-in. deep. Top —supported on cork. Bottom— supported on wood fiber acoustical material

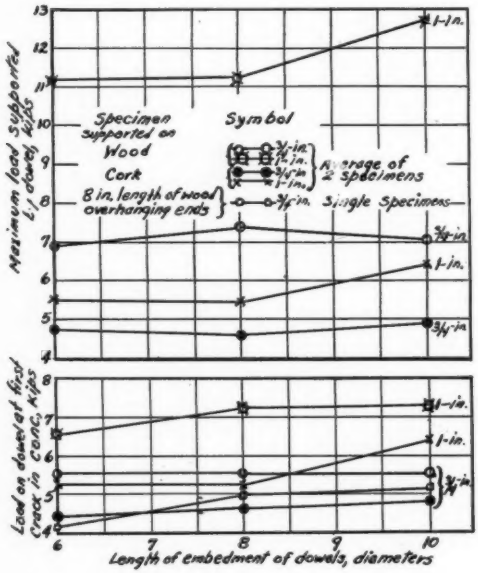
Maximum load supported by dowel, kips
Specimen supported on Wood Cork
8 in. length of wood overhanging ends
Symbol
Average of 2 specimens
Single specimens
Load on dowel at first crack in conc, kips
Length of embedment of dowels, diameters

Fig. 7—Effect of length of embedment on load-carrying capacity of ¾- and 1-in. dowels in plain concrete blocks 8 in. deep

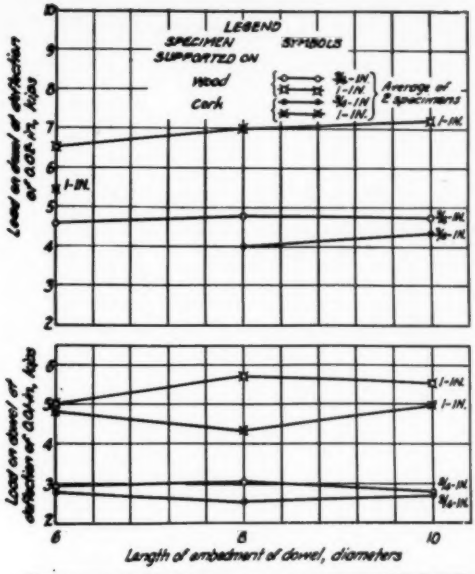

Fig. 8—Effect of length of embedment of ¾- and 1-in. dowels in plain concrete blocks 8-in. deep on deflection of dowels under load

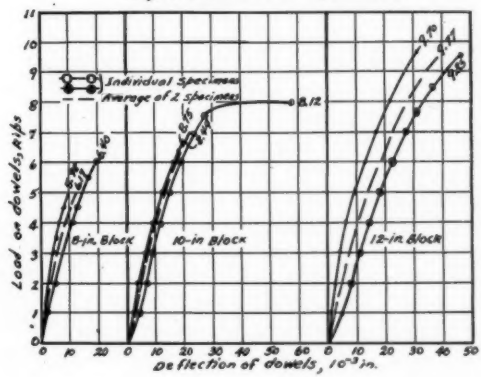

Fig. 9—Deflection of 1 x 10-in. dowels tested in plain concrete blocks supported on wood fiber acoustical material

ultimate P_u of samples resting on a rigid subgrade may be considerably higher than the load P_u of the samples supported by a soft base; (4) an increase of the width of block beyond the ratio $b/h = 3/2$ does not seem to have a definite influence on crack loads; (5) the most effective types of additional reinforcement are located close to the dowel, are able to withstand both vertical and horizontal forces and to distribute dowel loads over a larger distance from the dowel axis.

Causes of cracking or rupture

The load acting on a dowel is carried first on the concrete slab and then the

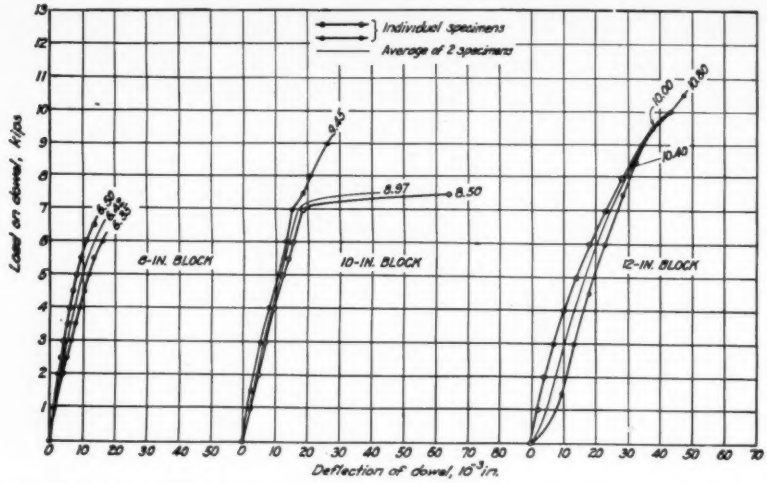

Fig. 10—Deflection of 1 x 10-in. dowels tested in plain concrete blocks supported on cork

supporting subgrade. The concrete is subjected to two types of stresses: (1) compressive bearing stresses in the contact area between dowel and concrete, and (2) longitudinal and transverse bending and shear stresses distributed over the whole concrete body.

The method used by Friberg for the analysis of bearing stresses is based on the assumption that the dowel is acting like a rod supported by a row of small concrete columns, each one being subjected only to a vertical pressure, p, and shortened by an amount y directly proportional to this pressure and independent of the deformation of neighboring columns. In reality, the supporting elements form a plain body in which this bearing pressure is balanced by three dimensional normal and shear stresses, dependent upon dimensions, elastic properties, boundary conditions of the body and compressibility of the foundation.

To understand the mechanism of cracking or failure of various types of test specimens, a careful analysis of the state of stress and strain in the con-

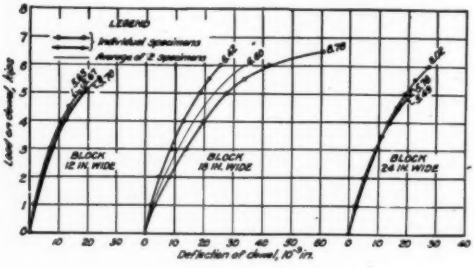

Fig. 11—Deflection of ¾ x 6-in. dowels tested in plain concrete blocks 8 in. deep supported on wood fiber acoustical material

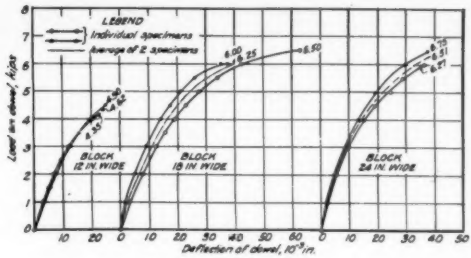

Fig. 12—Deflection of ¾ x 6-in. dowels tested in plain concrete blocks 8 in. deep supported on cork

crete blocks was necessary.[7] It was shown that the primary cause of radial cracks around the dowel and other cracks in the top or bottom surface of the blocks is a principal tensile stress f_t about equal to $10\sqrt{f_c'}$.

Allowable dowel loads

Fig. 5 and 6 indicate that as soon as the concrete begins to crack, the dowel deflection at the face of the joint increases rapidly. The breaking point, corresponding to crack load, P_c, has the same relation for concrete that yield point has for steel. Any load increase beyond this point produces permanent deformations of considerable magnitude and subsequently misalignment of the dowels and deterioration of the dowel groove. For this reason it is desirable to limit the allowable design load to about $3/5\ P_c$.

Tests on specimens without reinforcement can be summarized in the approximate formula

$$P_c = \frac{3}{2}\,\frac{d(h-d)}{1+\frac{e}{l}} \times f_t \qquad (4)$$

where

d = diameter

l = embedded or effective length of dowel

e = distance between the point of application of the load and the face of the joint, *i.e.*, the half joint width

$f_t = \sqrt{10\,f_c'}$ the tensile strength of the concrete

A reasonable value for the allowable load is

$$P_{all} = \frac{3}{5}P_c = \frac{9}{10}\,\frac{d(h-d)}{1+\frac{e}{l}}\,f_t \qquad (5)$$

Table 7 gives the values of P_{all} for various values of e and l for a 3500 psi concrete.

To check whether the dowel would be overstrained by the load, P_{all}, this load is compared with another design load, P_{ds}, based on the yield stress f_y of steel and determined by the approximate formula

$$P_{ds} \leqq \frac{3}{5} \times \frac{f_y\,\pi\,d^2}{32\left(\frac{e}{d}+\frac{5}{72}\right)+3} \qquad (6)$$

TABLE 7—ALLOWABLE CONCRETE LOAD

Dowel size, d, in.	Eccentricity, e, in.	Embedded length, l, in.	Allowable load, psi* Depth of concrete, h, in. 6	8	10
¾	⅜	4½	1935	2675	3410
		6	1975	2725	3480
		7½	2000	2760	3520
		9	2015	2780	3550
	½	4½	1890	2610	3325
		6	1935	2675	3410
		7½	1965	2715	3465
		9	1990	2745	3500
	⅝	4½	1840	2545	3245
		6	1900	2625	3350
		7½	1935	2675	3410
		9	1960	2710	3455
1	⅜	6	2510	3510	4515
		8	2545	3560	4580
		10	2570	3595	4620
		12	2585	3615	4650
	½	6	2460	3445	4425
		8	2510	3510	4515
		10	2535	3550	4565
		12	2560	3580	4605
	⅝	6	2415	3380	4345
		8	2470	3460	4450
		10	2510	3510	4515
		12	2530	3545	4560
1¼	⅜	7½	3010	4280	5550
		10	3050	4335	5615
		12½	3070	4365	5660
		15	3085	4385	5685
	½	7½	2965	4215	5465
		10	3010	4280	5550
		12½	3040	4325	5605
		15	3060	4350	5640
	⅝	7½	2920	4150	5380
		10	2975	4230	5485
		12½	3010	4280	5550
		15	3035	4315	5595

$$*P_{all} = \frac{9}{10} \times \frac{d(h-d) \times f_t}{\left(1+\frac{e}{l}\right)} \qquad f_c' = 3500 \text{ psi} \quad f_t = 592 \text{ psi}$$

TABLE 8—ALLOWABLE LOAD FOR DOWEL HAVING YIELD STRESS f_y

Dowel dia, d in.	Eccentricity e, in.	$*P_{da}$, lb, when f_y = 40,000 psi	50,000 psi	60,000 psi
¾	⅜	1999	2498	2974
	½	1597	1997	2377
	⅝	1330	1663	1979
1	⅜	4379	5473	6568
	½	3553	4442	5330
	⅝	2990	3737	4484
1¼	⅜	7949	9936	11923
	½	6537	8172	9806
	⅝	5552	6939	8327

$$*P_{da} = \frac{3}{5} \times \frac{f_y \times \pi\, d^2}{32\left(\frac{e}{d}+\frac{5}{72}\right)+3} \text{ lb.}$$

Table 8 gives the values for P_{da} corresponding to various dimensions of d and e for (1) a medium grade steel with f_y = 40,000 psi, (2) a hard grade steel with f_y = 50,000 psi, and (3) a still harder steel with f_y = 60,000 psi.

The magnitude of the eccentricity, e, has considerable effect on the dowel stresses and the bearing capacity is also dependent on the quality of steel. If a high-yield steel is used, then the concrete strength will be the essential factor governing the bearing capacity.

BOND RESISTANCE OF COATED DOWELS

The free ends of dowels bridging transverse joints of pavements are supposed to slide without offering appreciable resistance to the opening and closing of the joints. To ensure this sliding the free end of the dowel is usually coated with a paint, grease or other substance to prevent a bond between dowel and concrete.

The efficiency of various coatings was studied by measuring the slip in pull-out tests. The concrete blocks were 6 x 6 in. in cross section with a length equal to 8, 10 or 12 times the diameter of the embedded bars. Concrete strength, f_c', ranged from 3310 to 4070 psi.

Test procedure

The specimens were placed on a hemispherical bearing block having a hole in the center and the load was applied to the lower end of the bar. The slip of the bar was measured at the upper or free end of the bar with a 0.001-in. micrometer dial supported on the upper surface of the concrete prism adjacent to the free end of the rod.

TABLE 9—RESULTS OF PULL-OUT TESTS

Diam. of bar, in.	Sample No.	Type of coating	Length of embedment, in.	Max. load, lb	Load at slip of 0.25 in., lb	Max. ave. bond stress, psi	Condition of bar after test
¾	1	Uncoated (cold-rolled bar)	7	300	280	21.50	
			7½	660	650	37.35	———
			9	480	480	22.64	
¾	2	Zinc chromate primer	6	1010	170	72.38	Paint partly stripped off and adhering to concrete
			7½	1880	770	102.88	
			9	1690	950	79.69	
1	3	Zinc chromate primer	8	2710	1180	107.83	Paint partly stripped off and adhering to concrete
			10	5000	1340	159.15	
			12	7570	2700	200.80	
¾	4	Babbitt	6	2820	700	202.10	Coating partly stripped off; remainder peeled readily
			7½	2690	1450	152.22	
			9	7840	4860	369.71	
¾	5	Zinc	6	2050	440	146.91	Coating cracked transversely and segments moved along the bar during test
			7½	2600	690	147.13	
			9	2240	510	105.63	
¾	6	Phosphor bronze	6	1360	300	97.46	Coating partly stripped off and peeled readily
			7½	1840	350	104.12	
			9	1610	260	75.92	
¾	7	Lead	6	1330	540	95.31	Coating partly stripped off and peeled readily
			7½	1820	770	102.99	
			9	1820	540	85.83	
¾	8	Uncoated (plain hot-rolled bar)	6	3330	2300	238.65	
			7½	3720	2550	210.51	
			9	7210	5220	340.00	———
1	9	Uncoated (plain hot-rolled bar)	8	8000	3100	318.31	
			10	8960	3350	285.20	———
			12	11320	3880	300.27	

Test results

Results are summarized in Table 9 which also gives bond stresses corresponding to the greatest pull. The bond resistance of smooth, cold-rolled, uncoated bars was considerably smaller than the resistance of hot-rolled uncoated bars. Both types of uncoated dowels developed their greatest bond strength just before the "first slip" and their resistance decreased rapidly with continuing slip.

The ¾-in. bars with sprayed metal coatings also reached their maximum bond strength before the first slip. However, the pull required to produce this slip was many times greater than that of uncoated cold-rolled bars, but not much smaller than that of uncoated hot-rolled bars.

Bars with a zinc chromate primer, applied by dipping, develop their bond resistance with increasing slip. The final pull is ¼ to ⅔ of the greatest pull of the uncoated hot-rolled bars.

An ideal coating would act as a sleeve inserted between steel and concrete, but without adherence to either. In reality the condition is not fulfilled. For instance, the paint of samples 2 and 3 was partly stripped off the steel, but adhered to the concrete. The coating of samples 4, 5, 6 and 7 was also partly stripped off and the remaining crust could be peeled easily.

In any case there is no reason to expect that radial pressure exerted by the concrete on the skin of the coating and due to shrinkage of concrete shall

not offer any resistance to the slip of the bars. Actually it is only after this skin is torn that slip is possible.

The stripping of the coating started with a slip of only about 0.01 to 0.02 in., whereas the opening or closing of the expansion joint may require a slip of 0.03 to 0.80 in. If the coating should adhere to both concrete and bars, it would induce high tensile or compressive strains in the concrete. The deterioration of the coating prevents the generation of such strains, but also reduces the effectiveness of the protection of the bars against corrosion.

ACKNOWLEDGMENTS

The author thanks the former design managers of the Bureau of Yards and Docks, Captain E. H. Praeger and Commander L. C. Coxe, for having promoted and supported the wide research work; Douglas E. Parsons and David Watstein, for the excellent performance of tests by the National Bureau of Standards; and F. Knoop and G. A. Cunney of the Research Division of the Bureau of Yards and Docks, for their valuable help in preparing the report.

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Discussion of a paper by Henri Marcus:

Load Carrying Capacity of Dowels at Transverse Pavement Joints*

By P. C. DISARIO and AUTHOR

By P. C. DISARIO†

A review of Dr. Marcus' excellent paper should produce stimulating discussion among structural engineers who are continually faced with a similar problem in designing column bases with structural anchor bolts embedded in concrete. Fully aware of the intended application of Dr. Marcus' presentation, the writer would like to inject into the discussion the problem of allowable horizontal or shear load on anchor bolts embedded in concrete for holding down structural steel columns, chimney and process towers, equipment, etc. It would appear application of Dr. Marcus' formula should produce a satisfactory answer for all sizes of anchor bolts and strengths of concrete. However, the problem of anchor bolt design is greatly complicated by the unknown factor of initial tension produced by tightening of anchor bolts during construction. This initial tension through friction reduces the actual shear or horizontal load transmitted to the bolt.

Fig. A shows a group of curves of allowable shear values for anchor bolts of various sizes developed by several engineering organizations (curves *OH*, *WM*, *PD*); a plot of values from the Los Angeles Building Code (1943) for anchor bolts grouted in hollow masonry units (curve *LA*); and an extension of Dr. Marcus' formulas (curves *MS* and *MC*). These curves, except for the Los Angeles curve are based upon $f_c' = 3000$ psi, $f_y = 50{,}000$ psi, and for Dr. Marcus' formula $h = 6$ in., $e = \frac{3}{8}$-$\frac{1}{2}$ in. and $l > 8d$. It can be seen that allowable values employed by the engineering organizations are on the average much greater than the values obtained using Dr. Marcus' formula for allowable load on concrete (curve *MC*). The higher values have been used for many years and in most cases were probably arrived at by experience or approximation.

If we were to assume during the tightening process of anchor bolts in the field that a tension equal to the allowable tensile strength in each bolt was achieved and further, a coefficient of friction $u = 0.2$, then composite curve *X* which is obtained by adding this friction force and the allowable load from the plot of Dr. Marcus' formula (curve *MC*) is the result. It should be under-

*ACI JOURNAL, Oct. 1951, *Proc.* V. 48, p. 169. Disc. 48-13 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 24, No. 4, Dec. 1952, Part 2, *Proceedings* V. 48.

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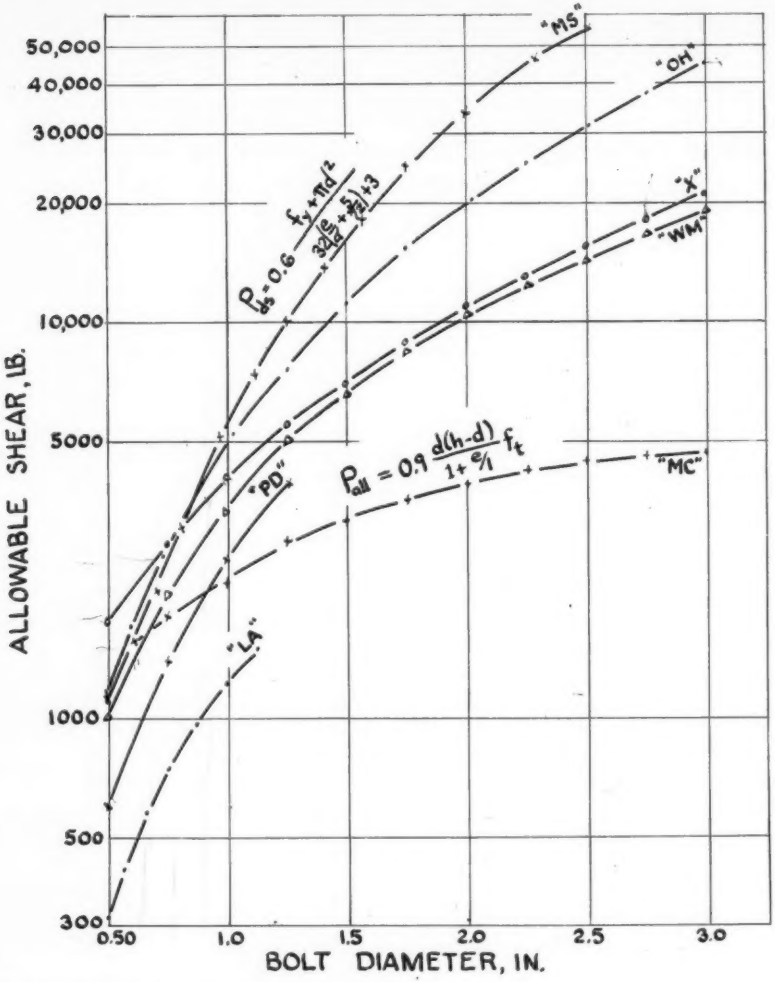


Fig. A—Data on allowable shear on anchor bolts
Curves OH, WM, PD—values employed by private engineering organizations
Curve LA—from Los Angeles Building Code (1943)
Curve MS—permissible shear on bolt, $e = \frac{3}{8}$ in., $f_y = 50{,}000$ psi
Curve MC—permissible shear on concrete, $e = \frac{3}{8}$-$\frac{5}{8}$ in., $f_c' = 3000$ psi, $l > 8d$, $h = 6$ in.
Curve X—composite curve combining curve MC and friction factor

stood that this is a simplification of the problem for discussion purposes only. Actually, the bolt feels only the amount of shear given in curve MC but for simplification and bolt selection during design the type of presentation given in curve X is usually made.

It is not inconceivable that normal tightening of anchor bolts in the field will produce a tensile stress within the bolt equal to the allowable stress. Evaluation of this factor and its resultant effect upon the permissible shear on anchor bolts has not been determined for structural anchor bolts other than by experience and judgment as far as the writer knows. However, in the case of bolting of flanged pipes it has been reported that field tests of such bolts indicate tensile stresses greater than twice the allowable for normal installations. Since yielding must necessarily occur at such high stresses, it appears reasonable to propose for design that under normal field conditions, anchor bolt tightening produces a tensile stress within anchor bolts equal to the allowable tensile stress. Curve X, when compared with other design curves given in Fig. A appears to confirm this point.

On page 181 formula $f_t = 10\sqrt{f_c'}$ is also written $f_t = \sqrt{10\ f_c'}$. In addition on page 182, Table 7, the allowable load is given as psi rather than lb.

AUTHOR'S CLOSURE

The author has been asked frequently whether empirical Eq. 4 might be used to determine the lateral resistance of anchor bolts embedded in concrete.

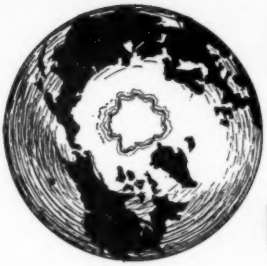
The answer is that the state of stress and strain in the concrete surrounding dowels is in many respects different from the state of stress and strain in the vicinity of anchor bolts. If we have the same spacing b for the dowels as for the bolts and the same effective concrete depth d, the resistance of the bolts must be considerably greater than the bearing capacity of the dowels.

The greater strength of anchor bolts is due not only, as assumed by Mr. Disario, to frictional resistance resulting from tightening, but also, and in a much larger measure, to the co-action of the horizontal steel plates forming the column base. These plates transmit and distribute a considerable part of the lateral forces by means of bond stresses acting over a large area of the interface of plate and concrete; thus both bearing stresses around the bolt and tensile stresses producing the diagonal cracks in the concrete are reduced.

Our tests have shown that ¾-in. dowels, 6 in. long, embedded in 8 in. deep concrete blocks reinforced by a steel plate ¼ in. thick, 10 in. long, 2 in. deep had a crack load P_c = 9650 lb and an ultimate load P_u = 14,670 lb, whereas the bearing capacity of similar test specimens without reinforcing plates was limited to P_c = 4450 lb and P_u = 4750 lb.

Taking into account the co-action of the horizontal plates, we recognize that bearing capacity of anchor bolts is dependent more upon the steel than upon the concrete stresses and that therefore Eq. 6 gives a more realistic evaluation of the design load than Eq. 5 and justifies the allowable loads as indicated by curve OH.

The author thanks Mr. Disario for having noted that the right equation for tensile strength of concrete is $f_t = 10\sqrt{f_c'}$ (and not $\sqrt{10\ f_c'}$ as written on page 181) and that the allowable loads indicated in Table 7 (page 182) are measured in lb and not in psi.



Letters from Readers*

BY WAY OF SYNOPSIS

A READER asks advice on overcoming cracking in tunnel linings.

H. S. MEISSNER tells an interesting story about reinforced concrete.

A method of determining moments in irregularly loaded beams is described by JOSEPH ZALKIN.

FRANK SLEETER wants information on building masonry walls in cold climates.

Cracks in Tunnel Lining (LR 48-10)

Cracking of the concrete lining of copper mine tunnels has created a troublesome problem in the solution of which the experience of others may be helpful.

Before taking out any body of ore, all necessary concrete tunnel lining is done 8 to 12 months in advance to allow the concrete to thoroughly harden before being subjected to any condition which might result from blasting in combination with the pressure of the rock where the mining is done at 1500 to 2000 ft below the surface.

The concrete is mixed in a conventional 3½-cu ft batch mixer and is dumped directly into a blowing machine which blows it through a 6-in. pipe to the forms by a sudden blast of 90-lb compressed air. In some instances there is definite evidence that the rock is under pressure while concrete is being placed. As a result the concrete is in some instances under pressure before full strength is reached.

Concrete deterioration usually begins about one month after mining operations start. Spalling usually shows in the roof slab first in the form of hair-line cracks. These widen and lengthen progressively, eventually extending down the walls to the bottom. More cracks appear until eventually the drift appears like a concrete brick lined opening. The blocks or slabs begin to fall off, undoubtedly shaken loose by blasting.

Reinforcing the concrete has been tried but the vibration from blasting causes cracks to appear in the concrete along the reinforcing bars and as a result speeds the disintegration process.

Samples of the concrete examined are cracked along various planes and the aggregate is not strongly bonded to the mortar. Uncracked samples have a compressive strength of 3000 psi. Clean river aggregate is used with a mix proportion of 1 sack of cement to 4 cu ft of aggregate. The mixing water is also clean.

In some places, where no rock movement or pressure develops due to location or type of rock, similar concrete lasts indefinitely. Some drifts concreted in 1946 are still in excellent condition. The opinion is that movement of the ground due to rock pressure during the hardening of the concrete breaks the bond and subsequent blasting serves to complete its deterioration.

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The writer would appreciate learning the experience of others with this problem and methods used to assure sound, durable concrete.

READER

Reinforcing—What is it? (LR 48-11)

A sidewalk in my home town, Ogden, Utah, put in shortly after the turn of the century when reinforced concrete was first coming into use presents an interesting answer to this question.

My attention was first attracted to this sidewalk because I thought (and quite likely so) I saw evidence of alkali-aggregate reaction in the concrete. Then, my curiosity was aroused when I noticed several large, bright, shiny rings of metal about 3 ft in diameter showing up with regularity all over the sidewalk. Knowing that on the site, years ago, an implement store had burned down I reconstructed in my own mind the following events.

Among the rubbish of this fire were many pieces of farm machinery, probably mowers, binders, hay rakes, etc. When the implement store was rebuilt the architect called for a reinforced sidewalk over the basement extending under it. The contractor, I surmise, decided, "Well, if its reinforcement he wants, I can give him plenty." So, in addition to bar steel, he threw in the damaged farm machinery, distributing the steel wheels with regularity in the fresh concrete over the whole slab. In Fig. 1 you can make out the rims of several wheels and, in some, the spokes and hub. I traced out about a dozen in a 50-ft frontage.

This evidences to me that concrete sidewalks do wear under foot traffic, for I believe

Fig. 1—Outline of "wheel" reinforcing in sidewalk

the steel wheels must originally have been well under the screeded surface of the sidewalk and that the finishers would have seen to it that they were covered. I believe the sidewalk is at least 40 years old. Through the decades a ceaseless stream of pedestrians, at the expense of much shoe leather, has gradually but inexorably abraded the concrete surface to reveal the incarcerated remains of fire-seared machinery. What now sticks above the surface is not rusted, but kept to bright polish by the shuffle of countless feet.

I think I am right, although nobody at the property could verify this story. I believe the contractor to whom reinforced concrete was something new decided to "reinforce with a vengeance."

H. S. MEISSNER, research engineer, U. S. Bureau of Reclamation, Denver, Colo.

Determination of Fixed End Moments in Beams with Irregular Loading Without Use of Formulas from Handbooks (LR 48-12)

In analyzing indeterminate structures by moment distribution it is necessary to start with fixed end moments of loaded members. These are generally computed by formulas found in handbooks. However, handbooks do not cover all possible conditions of loading. In some cases it is possible to reproduce the effects on a beam of an irregular loading by combining two other conditions of loading for which formulas are available. When this can not be achieved or is found to be impractical, then the fixed end moments must be obtained by some other means.

A method, generally overlooked, of finding fixed end moments in beams for any kind of irregular loading is indicated on page 84, of the Cross and Morgan book *Continuous Frames of Reinforced Concrete*. The method, which is entirely independent of complicated formulas, makes use of false reactions. These,

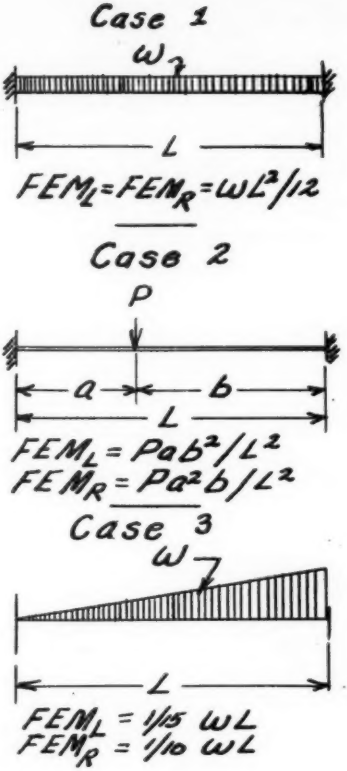


Fig. 1—Fundamental conditions of loading

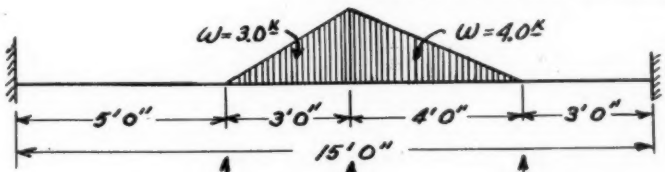
and end moments are computed. Next are computed the end moments in the beam with the false reactions applied as loads. The algebraic sum of the respective end moments thus obtained represents the fixed end moments of the given beam.

The only formulas required by this method are those shown in Fig. 1, for three fundamental conditions of loading.

To illustrate the application of this method, a problem has been worked out and is shown in Fig. 2. By introducing false reactions R_1, R_2, and R_3, there are formed four auxiliary spans. Distribution factors and fixed end moments for these are shown in lines 1 and 2, respectively. Balancing and distribution result in moments over supports shown in line 6. In line 7, are given shears at the supports for simple beam loading of spans. In line 8, are increments of shears resulting from unequal moments over adjacent supports. Finally, the actual shears and resulting reactions at auxiliary supports are given in lines 9 and 10, respectively. It should be noted that in this particular case moments over the fixed supports are positive.

The false reactions are shown applied to the beam as concentrated loads in Fig. 3. Here, are also shown computations for final fixed end moments at each end of the beam.

Fig. 2 (Below)—Application of method



		R_1		R_2		R_3		
D F		.37	.63	.57	.43	.43	.57	(1)
F E M	0	0	+600	−900	+1600	−1065	0	0 (2)
Balance	0	−222	−378	−400	−300	+457	+608	0 (3)
Distr.	−111	0	−200	−189	+228	−150	0	+304 (4)
Balance	0	+74	+126	−22	−17	+64	+86	0 (5)
−M	-74	−148	+148	−1511	+1511	−694	+694	+347 (6)
V_s	0	0	1000	2000	2670	1330	0	0 (7)
dV		+44	−454	+454	+204	−204	+347	(8)
V		44	546	2454	2874	1126	347	(9)
R			590		5328		1473	(10)

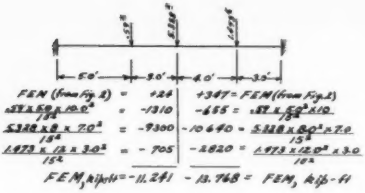


Fig. 3—False reactions applied as concentrated loads

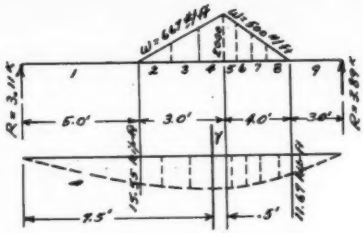


Fig. 4—Solution by column analogy

These are self explanatory and need no further elaboration.

As stated previously, this method lends itself to any loading and can be relied upon for a quick and easy solution of a problem whenever ready made formulas can not be found in handbooks.

Since this example cannot be checked by any handbook formulas a solution by "Column Analogy" is shown in Fig. 4 and Table 1. The two methods check very closely.

JOSEPH ZALKIN, engineer,
Corps of Engineers,
Washington District

TABLE 1—SOLUTION BY COLUMN ANALOGY TO ACCOMPANY FIG. 4

Col.	l	I	$a = \frac{L}{I}$	x	ax	$ax + lx$	$m s$	$p = m_s a$	x^1	N/x^1
						125.0				
1	5.0	1.0	5.0	−5.0	−25.0	10.4	7.775	38.875	−4.17	−162.0
						4.0				
2	1.0	1.0	1.0	−2.0	−2.0	.0835	17.0861	17.0861	−2.0	− 34.17
						1.0				
3	1.0	1.0	1.0	−1.0	−1.0	.0835	19.824	19.824	−1.0	− 19.82
						0				
4	1.0	1.0	1.0	0	0	.0835	21.565	21.565	0	0
						1.0				
5	1.0	1.0	1.0	+1.0	+1.0	.0835	21.74	21.740	+1.0	+ 21.74
						4.0				
6	1.0	1.0	1.0	+2.0	+2.0	.0835	20.095	20.095	+2.0	+ 40.19
						9.0				
7	1.0	1.0	1.0	+3.0	+3.0	.0835	17.222	17.222	+3.0	+ 51.666
						16.0				
8	1.0	1.0	1.0	+4.0	+4.0	.0835	13.5996	13.5996	+4.0	+ 54.4
						108.0				
9	3.0	1.0	3.0	+6.0	+18.0	2.25	5.835	17.505	+5.5	+ 97.0
			15.0		0	281.25		187.5117		+ 49.01

$$\frac{187.512}{15} - \frac{49.01 \times 7.5}{281.25} = \begin{matrix} 12.5 - 1.31 = 11.19 \text{ kip-ft} = FEM_L \\ 12.5 + 1.31 = 13.81 \text{ kip-ft} = FEM_R \end{matrix}$$

Masonry Walls in Cold Climates (LR 48-13)

We are designing several buildings to be built some time in the near future in Ontario. One of the questions concerns concrete masonry wall construction and the effect of a severe winter climate with its heavy freezing and thawing over a much longer period than in the States. One school of thought maintains that in a curtain wall using 10- or 12-in. concrete or cinder blocks, the dew point falling within the wall is bound to bring an eventual crumbling action that will make the wall short-lived. They hold that the proper wall for that climate is a 13-in. wall with a facing of either brick or stone and a backup of block, or preferably tile.

What is the practice of others under these conditions? Does condensation, or the threat of it, make an elaborate wall necessary?

FRANK SLEETER, director of plant engineering, Radio Corp. of America, Camden, N. J.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Prestressed concrete footbridge

The Reinforced Concrete Review (London), V. II, No. 4, Oct. 1950, pp. 210-211

Reviewed by C. P. SIESS

Brief description of a prestressed post-tensioned fixed-arch footbridge of 45-ft span built in the Oxford University Parks. A more complete description appears in *Concrete and Constructional Engineering*, V. XLV, Oct. 1950, p. 347.

Design of concrete mixes (in Czech)

E. WEINER, *Publications of the Faculty of Civil Engineering*, Czech Technical University, Prague, Reprint 33, 1951

Reviewed by IVAN M. VIEST

The author presents a method for the design of concrete mixes used at the Klokner Research Laboratories in Prague. This theoretical method is a combination of those proposed by Kennedy (ACI JOURNAL, 1950) and Bolomey (*Revue des Materiaux*, 1948).

Concrete casting in pressure shafts (Betongarbeider i trykksjakter)

B. LINDVIK, *Teknisk Ukeblad* (Oslo), V. 98, No. 23, June 1951, pp. 457-62

Reviewed by EIVIND HOGNESTAD

The 8-ft steel penstock in a 2000-ft pressure shaft of a hydroelectric power plant was installed and encased in concrete during the winter months of 1950-51. The slope of the shaft was 43 degrees, and most of the work was done from above. The concrete was successfully placed by means of form vibrators attached to the steel penstock and rod vibrators used by hand.

Plant for prestressed concrete

The Reinforced Concrete Review (London), V. II, No. 4, Oct. 1950, pp. 260-64

Reviewed by C. P. SIESS

Development work by the Field Test Unit of the Ministry of Works is described briefly. Methods of gripping and stretching wires for prestressed precast members, either pre- or post-tensioned, are described and illustrated. Reference is made to *National Building Studies Bulletin* No. 12.

Influence lines computation for redundant elastic arches (Calcolo delle linee di influenze degli archi elastici iperstatici)

CARLO CATALANO, *Giornale Del Genio Civile* (Rome), V. 88, No. 6, June 1951, pp. 370-377

Reviewed by GENNARO MIANULLI

The theorem of linear combinations is applied to redundant arch bridges for the determination of its influence lines. The method is illustrated by two practical examples showing both the analytical and graphical procedure.

Discussion and clarification on ferric and pozzolanic materials (Chiarimenti in tema di cementi ferrici e pozzolanici)

FABIO FERRARI, *Il Cemento* (Milan), V. 47, No. 9, Sept. 1950, pp. 148-151

Reviewed by GENNARO MIANULLI

The author's aim is to prove the results obtained by tests and studies on ferric and pozzolanic cements. He claims that his findings are conclusive and urges cement industries to investigate its possibilities commercially.

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Construction of a grain silo in Iraq

G. A. Pitt, *The Reinforced Concrete Review* (London), V. II, No. 4, Oct. 1950, pp. 214-246

Reviewed by C. P. Siess

Construction of a concrete silo by the slip-form method is described with particular attention to the concrete work and the details of the forms and construction procedures. Special mention is made of proposed methods for concreting in extremely hot weather.

The pull-out test (Utdragsprovet)

J. Osterman, *Betong* (Stockholm), V. 36, No. 1, 1951, pp. 1-8

Reviewed by Eivind Hognestad

The author indicates that a great number of pull-out tests have been reported in the literature, the results of which are not conclusive. A working hypothesis is presented, therefore, to attempt establishing the important factors to be investigated in the future.

Analysis of reinforced concrete rectangular and triangular flat plates restrained along their edges (Contributo al calcolo delle piastre di cemento armato rettangolari e triangolari vincolati al contorno)

A. Danusso, *Il Cemento* (Milan), V. 47, No. 9, Sept. 1950, pp. 152-156

Reviewed by Gennaro Mianulli

The author treats uniform loads on rectangular and triangular flat plates. He discusses the theoretical limitations and assumptions are made to arrive at a practical solution. Concentrated loads are also investigated; the complication in the analysis is discussed and a method of attacking this problem is given.

Swedish highway bridges in prestressed concrete (Svenska landsvagsbroar av forspand betong)

R. Kolm and J. Osterman, *Betong* (Stockholm), V. 36, No. 2, 1951, pp. 57-86

Reviewed by Eivind Hognestad

Advantages and disadvantages of various methods of prestressing are discussed and some Swedish highway bridges are described which are completed or under construction. In this connection the results of tests on two 15-ft prestressed beams are presented. These beams were first subjected to one million repetitions of working loads, then tested to failure in a static short-time test. Some plans for future bridges are also discussed briefly.

Simplified mechanics and strength of materials

Harry Parker, John Wiley & Sons, New York, 1951, 275 pp. $4.00.

An elementary treatment of the subject for use by practical men interested in mechanics and construction. Using simple arithmetic and algebra, emphasis is on material bodies in which there is no motion. After a thorough general survey of all the mechanical forces to be considered, their specific applications to columns, rivets and welds, shafts, pipes and tanks, concrete, and retaining walls and dams are discussed.

A wide variety of illustrative problems are included. Tables of allowable stresses, properties of sections, and others giving useful engineering data are provided.

Stable concrete mixes (Stabila betongblandningar)

S. G. Bergstrom, *Betong* (Stockholm), V. 36, No. 1, 1951, pp. 9-34

Reviewed by Eivind Hognestad

Deformability measurements are used to investigate the conditions governing segregation of concrete during transport and compaction. Segregation will be absent if the concrete mix is stable, *i.e.*, if its deformability remains constant over a reasonable period during compaction. The presence of segregation is indicated by decreasing deformability, and a marked increase in deformability indicates poor workability. Various methods of stabilizing concrete mixes are discussed in the light of experimental evidence.

Beam frame bridges for narrow roads (Balkrambroar for smala vagar)

A. Cronholm and J. M. Monsted, *Betong* (Stockholm), V. 36, No. 2, 1951, pp. 107-W

Authors' Summary

For spans of 40 to 80 ft, the most usual type of Swedish concrete bridges is the two-hinged beam frame with two or more main frames. This paper presents an alternative performance with only one main frame, especially adapted for narrow roads. A comparison between an actually constructed bridge of this type and an equivalent bridge of traditional design shows that the former is more economical. The bridge is in either case designed for a loading of one 6-ton axle or three 5-ton axles (spaced 16 and 4 ft apart).

Construction of a road in prestressed concrete

Civil Engineering and Public Works Review (London), V. 46, No. 538, Apr. 1951, pp. 254-255
Reviewed by CHARLES W. DOHN

Describes the construction of prestressed concrete road at the new town of Crawley, Sussex, England, the first road of this type in the country. The concrete slab is 400 ft x 6 in. and prestress is 200 psi by Freyssinet type cables in diamond pattern. Exact measurements of movement and strain will be made and it is possible that the degree of expansion will be sufficiently small to enable use of normal expansion joints.

Rapid analysis for arch bridges (Calcolo rapido dei ponti ad arco)

DOMENICO PRIOLO, *Giornale Del Genio Civile* (Rome), V. 88, No. 6, June 1951, pp. 379-391
Reviewed by GENNARO MIANULLI

Part 3—Two hinged arches with thrust eliminated

Treats the case of two-hinged arches with thrust eliminated. The horizontal pressure is determined by means of an exact squaring of the integrals, contained in the expression of the H term. This squaring is made possible by the shape of the arch and the law of variation of the moments of inertia of the cross section.

Part 4—Stability of piers

In the fourth installment, a rapid approximate method is shown to verify the condition of stability of piers in multispan arch bridges, assuming that each pier acts as a unit with its two adjacent arches.

Construction of a large sewage digester of 6100 cu m (215,430 cu ft) capacity in reinforced concrete (Bau eines groszen Faulbehalters in Stahlbeton von 6100 m³ Inhalt)

FRIEDRICH KIESZ, *Der Bauingenieur* (Berlin), V. 26, No. 4, Apr. 1951, pp. 97-103
Reviewed by ARON L. MIRSKY

The author describes the structural design and construction of the large closed sewage digester in Wuppertal. The digester is 23 m (75.5 ft) in diameter at about midheight and 23.61 m (77.5 ft) in total height, about half being in the form of an inverted cone below ground level. The domed digester was adopted to collect the sewage gas for useful purposes.

Comparisons with two other digesters are made, showing the present design to be economical of steel and concrete in terms of sewage capacity.

Remarks on the theory of plates and flat slabs (Notes a propos de la theorie des plaques flechies et des planchers-champignons)

R. L'HERMITE, *Annales de l'Institut Technique du Batiment et des Travaux Publics* (Paris), No. 192, June, 1951
Reviewed by PHILLIP L. MELVILLE

A theoretical study of slabs simply supported on two opposite edges and with additional supports from columns in rows parallel to the bearing edges. Timoshenko's theory of plates is expanded by the method of images and relaxation method. An example is worked out showing that it is possible in certain cases of loading to make openings in a slab by the use of appropriate correction factors.

Suggested principles of low-pressure steam-curing of mortar and concrete

A. G. SAUL, *Cement and Lime Manufacture* (London), V. 24, No. 3, 1951, pp. 39-44
Reviewed by R. P. VELLINES

The author briefly summarizes and compares the work of other investigators of the effect of steam curing of cement and concrete at atmospheric pressure on compressive strength. The apparently conflicting recommendations of these investigators are attributed to differences in test procedures and experimental data, graphically presented, are offered in reconciliation. The scope of the author's tests is indicated by 31 different treatments of six cubes of each of two mixtures with "rapid hardening," "ordinary" and special low-alumina portland cement used at two water-cement ratios.

Concrete beams of trapezoidal cross section (Betongbalkar med trapetsformad tvarsektion)

U. ALESUND, *Betong* (Stockholm), V. 36, No. 1, 1951, pp. 35-56
Reviewed by EIVIND HOGNESTAD

A theoretical investigation of symmetrically loaded reinforced concrete beams with trapezoidal cross section is presented. The necessary equations are developed on the basis of the straight-line theory as well as on an ultimate basis.

It is shown that, for the same strength, depth and tension reinforcement, the concrete consumption of a trapezoidal section with the smaller side in tension is less than for a rectangular beam. The author also indicates that this trapezoidal section is superior to the rectangle with respect to cracking.

New method for accelerated testing of cement hardening (in Czech)

K. WAITZMANN, *Stavebni prumsyl* (Prague), June 1951, V. 1, No. 11, pp. 252-257

Reviewed by IVAN M. VIEST

Outlines a simple method of accelerated testing of cement paste for tensile and compressive strength and presents a correlation of test results from the accelerated and conventional 7-day tests. The accelerated test is made with small cement beams boiled for three hours 24 hours after mixing the paste. The test results are uniform and the correlation good.

Understanding and computing safety factor in reinforced concrete construction (La conception et le calcul du coefficient de securite dans les constructions en beton arme)

M. ROS, F. CAMPUS and E. TORROJA, *Annales de l' Institut Technique du Batiment et des Travaux Publics* (Paris), No. 194, June 1951

Reviewed by PHILLIP L. MELVILLE

A detailed study backed by numerical examples of the determination of factor of safety in concrete construction. It is considered as a ratio applied to stresses rather than loads. The crushing strength is used for concrete and is defined as the result of five tests out of six. The elastic limit is used for steel. The probability of complete or partial failure has been evaluated with its resulting cost to calculate the opimum factor of safety by an equation relating the cost of construction to the cost of the damage at failure. This calculation requires the use of uncertain statistical data.

Application of plastic theory to bending of centrally loaded columns (in Hungarian)

A. SZECHY, *Magyar Kozlekedes Mely-es Vizepites* (Budapest), V. 1, No. 5-6, May-June 1949, pp. 317-324

HUNGARIAN TECHNICAL ABSTRACTS

Deals with the problem of. calculating centrally loaded, hinge supported columns of uniform circular cross section and determination of the shape of the axis of buckled supports. The buckling stress values, which must be taken into account when applying cross sections of various shapes to an identical material, display substantial deviations. Experiments and calculations reveal that Euler's formula can be applied only in cases of high loads. Curves featuring various building materials may be substituted, in part and in sections, by straight lines, except for curves symbolizing concrete. This method of calculation may be applied to open U- and T-shaped cross sections.

Lifting procedures with pressure injections of cement grout (Hebungsverfahren mit Zementeinpressungen)

WALTER BERNATZIK, *Der Bauingenieur* (Berlin), V. 26, No. 4, Apr. 1951, pp. 107-112

Reviewed by ARON L. MIRSKY

In 1940 there was developed the idea of building heavy structures to act like pistons in a hydraulic cylinder; the cylinder itself was constructed of heavy steel plates on the ground and filled with sand, and in the event of unequal settlement of the structure the judicious use of water flow in the cylinder was to result in the correction of the differential settlement. During the last war a battery of coke ovens built in accordance with these principles was faultily constructed, so that the use of water alone only made matters worse; this article describes the use of pressure grout which was resorted to. Results at three stages of the work are shown on contour plans, illustrating the efficacy of the method.

Two prestressed concrete highway bridges in Sweden (Broar av forspand betong vid Stageborg och Kavlinge)

S. HILDEBERG, *Betong* (Stockholm), V. 36, No. 2, 1951, pp. 87-106

AUTHOR'S SUMMARY

In this paper, two Swedish bridges in prestressed concrete are described. One, the bridge over Slätbaken at Stegeborg on the east coast of Sweden about 200 miles south of Stockholm, is just completed. This bridge is built with 3 continuous head beams with the spans 19.3 + 25.3 + 25.3 + 19.3 m. The width of the bridge is 7.5 m. The head beams are prestressed according to the Freyssinet system. The cables consist of 12 wires 5 mm in diameter which are placed in a sheet tube and, after stressing, injected with cement mortar.

The other bridge, over the river Kävlingeån at Kävlinge in southern Sweden, is just getting underway. This bridge is 15 m wide and 75 m long. The superstructure has 5 continuous head beams with the spans 17.9 + 25.6 + 17.9 m, prestressed in the same manner as the bridge at Stegeborg. The cable goes from one end of the bridge to the other.

Buckling of reinforced concrete roadway bridge slabs with tied steel beams and the influence of one-sided heating of the composite bridge slabs (Das Knicken der Stahlbeton-Fahrbahnplatten bei gekoppelten Stahltragern und der Einflusz einer einseitigen Erwarmung der gekoppelten Fahrbahnplatten)

F. Dischinger, *Der Bauingenieur* (Berlin), V. 26, No. 4, Apr. 1951, pp. 103-106

Reviewed by Aron L. Mirsky

In *Der Bauingenieur*, V. 24, 1949, No. 11 and 12, the author described a method of constructing composite bridge decks using prestressed concrete slabs. The slab is placed on rollers resting on the supporting beams and prestressed, either by jacks reacting on abutments formed on the ends of the beams or by cables; the prestressed slab is then rigidly connected to the beams by riveting or welding previously provided connection details at the rollers.

In this article the author investigates analytically the buckling of the slab between these connection details, and the effect on stresses in the composite section of unequal heating. A numerical example is given. Also discussed is the doweling or connection of slab to beam and the stresses for which these connections must be designed.

The new Herdecke bridge over the Ruhr, a continuous composite girder bridge with prestress (Die neue Ruhrbrucke Herdecke, eine durchlaufende Verbundtrabergrucke mit Vorspannung)

H. Homberg, E. Kohling, and D. Fuchs, *Der Bauingenieur* (Berlin), V. 26, No. 5, May 1951, pp. 129-132; No. 6, June 1951, pp. 172-176

Reviewed by Aron L. Mirsky

Describes the conception, design, and construction of this bridge, which consists of three continuous spans (158—210.65—158 ft) on a slight skew. The jointless reinforced concrete deck, which is covered with an asphalt wearing surface, is forced to act in a composite manner with the welded girders by means of half-loop connectors welded to the top flanges. The deck is also anchored to the ends of the girders by special bars welded to the top flanges so that the prestress in the slab is effectively transmitted also to the girders.

The second part of the article includes curves of moments and of stress distribution in the composite section for various conditions.

It can be seen from this, if additional proof be needed, that European designers are far ahead of their American brethren in the effective use of prestressing.

Measurement of cement specimen lengths

Alfred Beitlich, *Rock Products*, V. 54, No. 6, June 1951, pp. 138 and 142

Reviewed by Donald M. Agrimson

Description of an accurate optical method for measuring minute changes occurring in cement specimens while curing.

The specimens, only $\frac{1}{16}$ x $\frac{3}{8}$ x 3 in., are provided with two small glass rods—one sealed to each end. The length measurements between the outer ends of the glass rods are taken by a micrometer slide graduated to 0.0001 in. Contact between the reference points of the specimen and the feeler points of the micrometer is detected by utilizing the fact that interference bands caused by light striking upon a very thin film are changed by the slightest variation in thickness of film. This film is produced by pressing a microscope cover glass against a microscope slide. These are inserted in the micrometer such that one of the glass rods will apply a very slight pressure when contact is made. This will change the thickness of the film and thereby produce a change of position in the interference lines. Illumination is provided by an electric light bulb operated by a flashlight battery. The light reflected from the glass plate is viewed by a magnifying lens. An excellent sketch is included in the article showing the top and side views of this optical device.

The lifting of the Autobahn bridge over the Saalach River near Bad Reichenhall (Die Hebung der Autobahnbrucke uber die Saalach bei Bad Reichenhall)

Otto Veit, *Der Bauingenieur* (Berlin), V. 26, No. 1, Jan. 1951, pp. 1-3

Reviewed by Aron L. Mirsky

This structure was built in 1937-38 as two adjacent curved-rib bridges of 90-m span with attached approach spans; each bridge consisted of three ribs with top slab and lower compression flanges.

In the last days of World War II the Germans dynamited the bridges at the crown, tilting the ends into the river. American engineer troops dynamited it further to reduce stream obstruction. Rather than blow up the remainder to clear the way for a new structure, the end portions (each weighing some 2200 tons) were raised

back into position and the destroyed portions replaced.

The author estimates that about 60 percent of the concrete in the original superstructures was retained, saving about ⅔ the cost of a new bridge to say nothing of the saving in scarce cement and reinforcement.

Full details are given. (A short description, with photographs, may also be seen in *Engineering News-Record*, Oct. 5, 1950, p. 52.)

Scour around bridges

Research Report No. 13-B, Highway Research Board, 1951, 22 pp.

Included in this report are two papers presented at the 30th annual meeting of the Board, Jan., 1951.

The first paper is a "Progress Report of Model Studies of Scour Around Bridge Piers and Abutments," by Emmett M. Laursen. It includes a statement of the problem of underscour and outlines plans for isolating and measuring the effects of the several variables involved. The four phases of the investigation are: (1) geometry of piers and abutments; (2) hydraulic characteristics of the stream; (3) sediment characteristics and (4) geometry of channel cross section and alignment.

The second paper describes the "Investigation of Flexible Mats to Reduce Scour Around Bridge Piers," by C. J. Posey, D. W. Appel and E. Chamness, Jr. In these studies it was found that the greatest hazards to the functioning of flexible mats were (a) the tendency of upward currents to lift sediments through the mat, and (b) the tendency of the mat to bridge or buckle due to bottom irregularities which caused underchanneling and dislocation of bed materials. The report includes discussion of requirements for effective protection of erodible beds and offers suggestions for further research.

Core boring on the roadway deck of the Cologne-Deutz bridge over the Rhine (Kernbohrung an der Fahrbahnplatte der Koln-Deutzer Rheinbrucke)

FRANZ BRAUN, *Der Bauingenieur* (Berlin), V. 26, No. 6, June 1951, pp. 187-188

Reviewed by ARON L. MIRSKY

The deck of this bridge is constructed of concrete 4.72 in. thick, tied to a thin flat steel plate forming the bottom surface by reinforcing-bar trusses running normal to the stringers and welded to the plate (somewhat on the order of the American "Cofar"). The whole thus forms a composite section, and is calculated as such.

Cracks in the concrete caused concern, although it was decided they were due solely to temperature and shrinkage effects. This article describes a single core boring made at one of the widest cracks, near a construction joint. The boring was centered among both transverse reinforcing trusses and longitudinal bars. It was not necessary to core through the flat metal plate as well, since the concrete separated from it (there being apparently little natural bond, or else due to the coring operation), although small bits of concrete remained attached to the plate at the welds attaching the reinforcement thereto.

The concrete was found to be sound. More importantly, it was found that the crack did not extend all the way through the thickness of the concrete, and there was no evidence of rust or corrosion on the steel plate. The author concludes that such cracks are not injurious to the structure.

Thermal expansion of concrete

D. G. R. BONNELL and F. C. HARPER, *National Building Studies*, Department of Scientific and Industrial Research, London

Reviewed by E. W. BAUMAN

Results of this investigation make a worthwhile contribution to the information available on this phase of concrete technology. Tests to determine the thermal expansion of concrete were conducted to obtain information as to the affect of aggregates, cement, and curing conditions.

Eight types of aggregates were employed for conventional concrete, as well as foamed slag representing the lightweight field: gravel, granite, quartzite, dolerite, sandstone, limestone, portland stone and blastfurnace slag. Three types of cement were involved: portland, high alumina and portland blastfurnace.

Coefficient of thermal expansion was made on the aggregate itself as well as the concrete. A rather unique method for measuring coefficient of thermal expansion of concrete was employed.

Results of the investigation justify the following conclusions: The coefficient of thermal expansion of a concrete depends

largely on the type of aggregate from which it was prepared—concretes with siliceous aggregates having the highest expansions; those with limestones the lowest; and those with igneous rocks and blastfurnace slag occupy an intermediate position. Further, the thermal expansion increases slightly as the cement content is increased; age appears to have only a minor effect on the expansion; and desiccated and water-saturated concretes are identical and somewhat lower in expansion than partly dried specimens.

Concrete almanac (Beton Kalender)

Wilhelm Ernst & Sohn (Berlin), 1951, 40th Ed. Part I, cloth, 656 pp.; Part II, paper, 400 pp. 16 DM.
Reviewed by Aron L. Mirsky

Although mainly a pocketbook on concrete, plain and reinforced, this work includes data on many other engineering subjects, as will be apparent from a mere listing of the contents:

Part I: mathematics and tables; materials of construction; strength of materials and theory of structures; concrete; and some 280 pages of DIN (German Industrial Standards) codes on design, loadings, cement, etc.

Part II: short articles, by various writers, on: various aspects of construction (including costs, forms, etc.); foundations and piles; retaining walls and quays; walls of buildings; slabs of various types; roofs; precast members (including prestressed); beam and frame and arched bridges; steel scaffolding; silos; and (some 55 pages) summaries of British, French, Italian, Russian, Spanish, and American codes on reinforced concrete (the U. S. A. being represented by ACI 318-47).

Both parts contain advertising pages in addition to text pages; yet the two together are less than 1½ inches thick, due to the good grade of paper used. The other dimensions (6 x 4¼ in.) are such that the books, singly or together, will fit in almost any coat pocket, although they might be lost among the ponderous tomes common in most technical libraries.

German-reading concrete men will find these books valuable, since they bring together, albeit in necessarily abridged form, most of the recent developments in concrete as well as a summary of concrete practice in the countries already mentioned. It is interesting to note that the Scandinavian countries are not included.

Symposium on foam concrete

Cement och Betong (Malmo), V. 26, No. 1, Mar. 1951, pp. 1-98
Reviewed by Eivind Hognestad

The foam concrete industry is only about 20 years old, which period is indeed modest compared to the use of concrete over centuries and of tile and bricks over millenniums. Foam concrete has been developed so rapidly in Sweden, however, that it today plays an important part in the construction industry of that country. The majority of the articles below pertains to Siporex, which is a foam concrete with portland cement as a binder.

The development of the foam concrete industry in Sweden

G. Helsten, pp. 3-8

The development of the foam concrete industry is studied on the basis of official industrial statistics. Production increased from about 10,000 tons to well over 300,000 tons in the period 1930-50. Presently the value of foam concrete masonry units exceeds that of normal concrete units by near 100 percent. Foam concrete is therefore firmly established as an important branch of the Swedish building industry.

Siporex through 15 years

B. Sjogren, pp. 9-14

The raw materials of Siporex are portland cement and quartz sand dust, the foam structure being obtained by the addition of aluminum powder, which generates hydrogen through chemical reactions. During high-pressure steam curing at temperatures over 150 C, a chemical reaction takes place between the cement and the sand dust, resulting in high strength and greatly reduced shrinkage. Siporex has gained a prominent place in the Swedish building industry during its 15 years of use.

Precast Siporex slab floors

A. Hallgren, pp. 15-30

The production of precast reinforced Siporex floor elements in Sweden is presently about 400,000 sq ft per year. The author describes the various types of elements and gives data regarding physical properties, design, construction and cost of Siporex slab floors.

Industrial roofs of reinforced Siporex slabs

A. Hallgren, pp. 31-53

During the last 15 years, about 50 million sq ft of roof surface consisting of reinforced

Siporex slab elements has been constructed in Sweden. This paper describes the various types of roof elements, and data regarding physical properties, design, and construction methods are presented. The author also discusses the problems relating to moisture condensation in some detail.

Prefabricated Siporex houses
R. WIDEGREN, pp. 57-66

A description of the construction of prefabricated homes consisting of a combination of reinforced Siporex wall slabs, floor units and roof elements.

Tests of walls made from Siporex masonry units
H. NYLANDER and H. A. VINBERG, pp. 67-97

A detailed report on laboratory tests of 46 walls made from Siporex masonry units.

Proceedings of the German Concrete Institute

Deutscher Beton-Verein, Wiesbaden, V. 47, 1950
Reviewed by PAUL ROGERS

The 1950 issue of the Proceedings of the German Concrete Institute contains numerous interesting papers, some of them presenting entirely new ideas.

The first major paper is by Dr. Dischinger, who, with Finsterwalder developed the Z-D system of thin-shell roof construction. In this article he advocates the use of shell design for arch dams. He starts with Jorgensen's theory of maintaining a constant angle of curvature for arch dams. Although this theory has already proved to be economical, he goes further and proposes that the downward increasing thickness of dams be made of separate laminations instead of continuous pours. Each lamination could be designed as a shell, cylindrical at the upper portion and rotational below. The load carrying capacity of shells is well known, and considerable economy and safety can be obtained by this method. The different laminations are separated by such media that the water pressure is transmitted without shear forces. Such media can be water, gravel, asphalt, or similar materials.

In following articles Veit describes the lifting of a highway bridge over the Saalach River. This reinforced concrete bridge of about 300-ft span was lifted out of the river bed, and by dint of special efforts and rather unusual construction methods, restored and reopened to traffic.

J. B. van Bruggen describes the construction details of a four-lane, double section tunnel under the Maas River in Rotterdam. The total length of the tunnel is about 3700 ft, of which about 1800 ft is under water. This underground section was composed of nine precast sections of about 200 ft each and sunk in the river to a previously prepared foundations. Due to the immense dimensions of these sections, special floating equipment was used, and unusual construction methods employed.

In a subsequent article Rüsch presented a very comprehensive and extensive study of the design of prestressed concrete members. This paper is followed by another by Vaessen describing the construction details and practices of precasting the archribs of open and solid spandrel arch bridges. Some of the spans are about 160 ft long, and the ribs, which are three-hinged arches, are cast on the ground and lifted and placed by huge gantry cranes.

Morsch presented a thorough study on the establishment of breaking moments of prestressed concrete joists and beams.

Two authors presented papers on the ultimate or limit design of reinforced concrete. Ros dealt with the problem of factors of safety and allowable stresses, while Bittner analyzed a system of design which eliminates the ratio of n.

Klett presented a rather extensive paper on the reconstruction problems of bridges and buildings under the authority of the Stuttgart railroad management. Numerous bridges, viaducts and railroad stations had been restored to traffic which involved considerable ingenuity on the engineer's part. One of their very interesting devices was the use of prestressed gable roofs. Such roofs ordinarily produce outward horizontal forces, which are resisted by tie-rods, piers, truss-action, or as part of gable rigid frames. By an ingenious prestressing these gables are self-anchored, thus eliminating outward forces; consequently they can be placed on any masonry supports.

The following paper by Hummel deals with lightweight concrete, while the last of the articles by Kleinlogel treats the problem of air-entraining concrete.